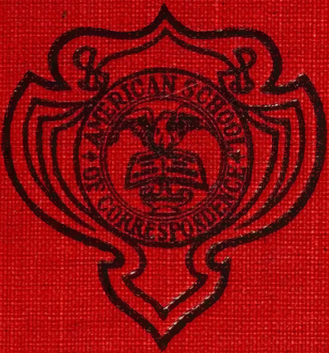
WATER SUPPLY

TURNEAURE



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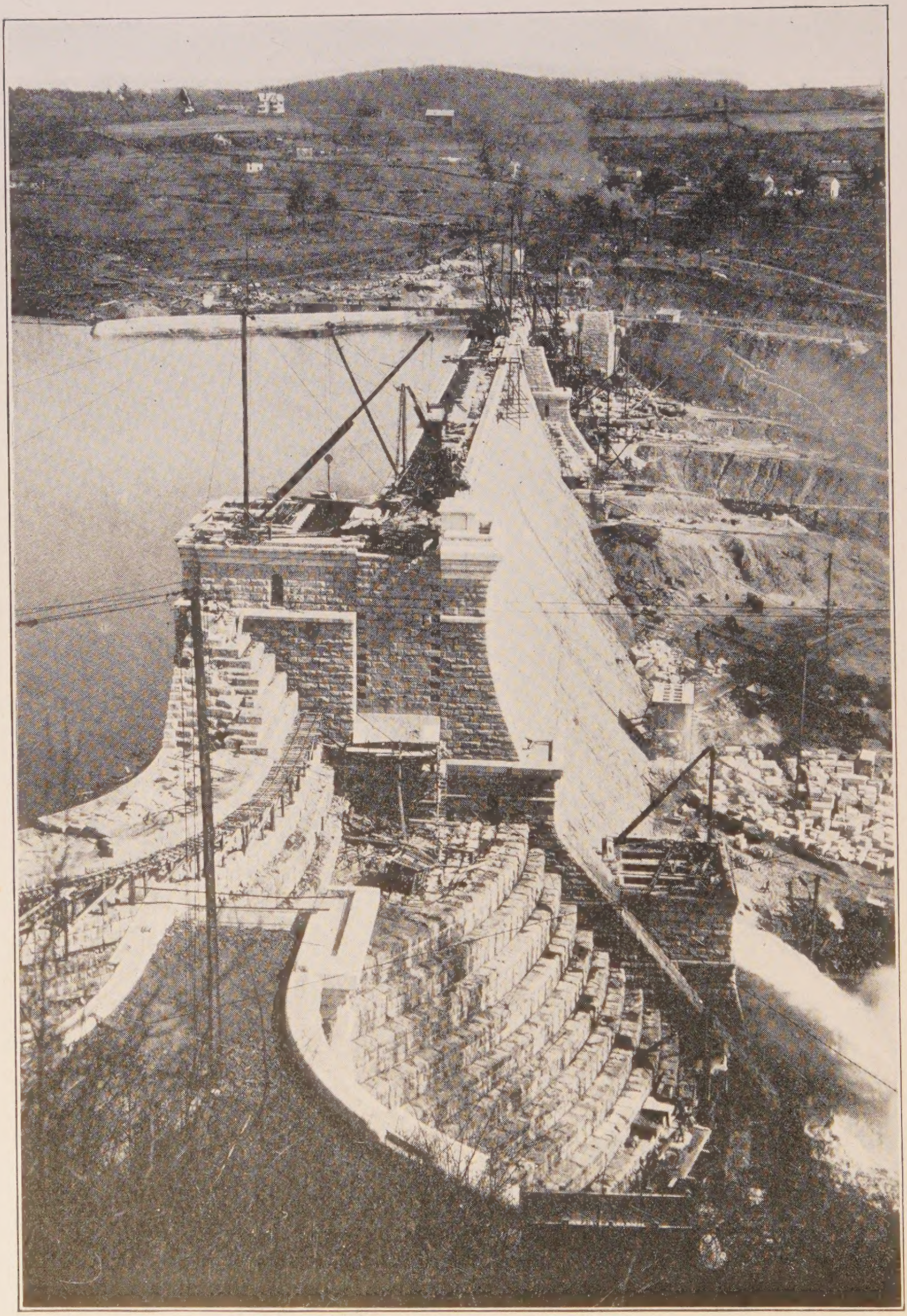

NEW CROTON DAM UNDER CONSTRUCTION

Largest masonry dam in the world, part of the waterworks system of New York City. Took 14 years to build, requiring about 850,000 cu. yds. of masonry. Completed in 1906. Cost, $7,700,000. Length, 2,400 feet; height, 301 feet; thickness, 216 feet at base, tapering to ten feet at top of spillway (at left) and 21 feet at top of main dam. Capacity, 30,000,000,000 gals., and with auxiliary dams 100,000,000,000 gals. Water at dam, 160 feet deep, the impounded river forming a lake 20 miles long and 2 miles in extreme width, burying under 30 feet of water the old dam 3 miles upstream.

Water Supply

A Treatise on the

SOURCES, DISTRIBUTION, AND CONSUMPTION OF WATER FOR COMMERCIAL AND DOMESTIC USES, AND MODERN PRACTICE IN THE CONSTRUCTION OF WATERWORKS AND PURIFICATION PLANTS

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ILLUSTRATED

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Foreword

IN recent years, such marvelous advances have been made in the engineering and scientific fields, and so rapid has been the evolution of mechanical and constructive processes and methods, that a distinct need has been created for a series of *practical working guides*, of convenient size and low cost, embodying the accumulated results of experience and the most approved modern practice along a great variety of lines. To fill this acknowledged need, is the special purpose of the series of handbooks to which this volume belongs.

❡ In the preparation of this series, it has been the aim of the publishers to lay special stress on the *practical* side of each subject, as distinguished from mere theoretical or academic discussion. Each volume is written by a well-known expert of acknowledged authority in his special line, and is based on a most careful study of practical needs and up-to-date methods as developed under the conditions of actual practice in the field, the shop, the mill, the power house, the drafting room, the engine room, etc.

❡ These volumes are especially adapted for purposes of self-instruction and home study. The utmost care has been used to bring the treatment of each subject within the range of the common understanding, so that the work will appeal not only to the technically trained expert, but also to the beginner and the self-taught practical man who wishes to keep abreast of modern progress. The language is simple and clear; heavy technical terms and the formulæ of the higher mathematics have been avoided, yet without sacrificing any of the requirements of practical instruction; the arrangement of matter is such as to carry the reader along by easy steps to complete mastery of each subject; frequent examples for practice are given, to enable the reader to test his knowledge and make it a permanent possession; and the illustrations are selected with the greatest care to supplement and make clear the references in the text.

¶ The method adopted in the preparation of these volumes is that which the American School of Correspondence has developed and employed so successfully for many years. It is not an experiment, but has stood the severest of all tests—that of practical use—which has demonstrated it to be the best method yet devised for the education of the busy working man.

¶ For purposes of ready reference and timely information when needed, it is believed that this series of handbooks will be found to meet every requirement.

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WATERWORKS PUMPING STATION AT ANDERSON, INDIANA

Here, in February, [illegible], were carried out tests which finally demonstrated the possibility of the absolute removal from water of all disease germs of animal or vegetable origin by means of a process using lime and sulphate of iron, with a small proportion of sulphate of copper. Over 40 cities in the United States have adopted this process for the purification of their water supply.

WATER SUPPLY.

PART I.

INTRODUCTION.

1. Historical. The earliest method of artificially obtaining a water supply was by the digging of wells. These were at first mere shallow cavities scooped out of the ground in low places; but it is interesting to know that the sinking of deep wells through rock dates from a very early period, the Chinese having been familiar with such work from very early times. Besides wells, other works for water-supply purposes were constructed by the Ancients, such as reservoirs, cisterns, aqueducts, etc.

The greatest development of waterworks construction in ancient times took place during the prosperous period of the Roman Empire, some of the finest works having been built at this time. To supply the chief cities of the empire great aqueducts were constructed, many miles in length, and there were in some cases several such aqueducts supplying a single city. Rome was at one time supplied from fourteen different aqueducts some of which had a length of 40 miles. The first of these was built about 312 B.C. and the last about 305 A.D. Some of the other cities which were well supplied with water at this time were Paris and Lyons in France, Metz in Germany, and Segovia and Seville in Spain.

The distribution of water in this age was by no means general. From the aqueducts the water first passed into large cisterns, and from these it was distributed through lead pipes to the fountains, baths and various public buildings, and to a few private consumers. The masses of the people were obliged to get their supply from public fountains. While the actual amount of water used by private consumers was not great the liberality of the supply for public purposes was so great that the total consumption was in many cases very high, some estimates making the consumption of water in Rome as high as 300 gallons per capita daily.

After the fall of Rome the entire subject of water supply was neglected for many centuries, and as one result, Europe was ravaged many times by terrible pestilences, due to polluted water. In some cases even the purpose for which the ancient aqueducts had been built was forgotten by the inhabitants.

The development of modern waterworks began in Paris and London as early as the beginning of the 17th century, but little progress was made until the application of steam to pumping engines, first made in London in 1761. Since 1800 the development has been very rapid, both in Europe and America.

The first works in America for the supply of water to towns were those of Boston, built in 1652. Machinery was first used for pumping water at Bethlehem, Pennsylvania, where the works were put into operation in 1754. The first use of the steam engine was at Philadelphia in 1800, and in New York steam was applied in 1804. The principal development in this country has taken place since 1850, about ninety-eight per cent of all existing works having been constructed since that time. Nearly all towns of 2,000 inhabitants or more now have a public water supply, and the construction of works is progressing rapidly in many smaller towns and villages. While there is more work yet to be done in this direction, the chief work of the future will be in providing increased supplies for the rapidly growing cities and towns of this country, in developing new and better sources of supply and in the improvement of the quality of the existing supplies. There is also much opportunity for the engineer in the management of waterworks, in the direction of reducing cost of operation, prevention of waste and in the improvement of service in many other ways.

2. Value and Importance of a Public Water Supply. The most important use of a public water supply is that of furnishing a suitable water for domestic purposes. For such use the prime requisite is that the water should be pure. The transmission of certain diseases such as cholera and typhoid fever by polluted water is now universally recognized, and the value to a city of a pure supply when compared to one constantly polluted by sewage can scarcely be overestimated.

Another highly important function of a water supply is that of furnishing the necessary flushing water for a sanitary system of drainage. The most satisfactory and economical method yet found for disposing of the organic wastes of a community is by the water-carriage system. Such a sewerage system is manifestly of but slight value to the public at large without the coexistence of a public water supply, as otherwise the necessary water for the flushing of closets—the most important function of a sewerage system—can be afforded by but few.

Besides furnishing an improved supply from the sanitary standpoint, a public works may often be made to furnish a water which for other reasons will be of greatly increased value to the domestic consumer; such as a soft water in place of a hard well water,—a point of very considerable importance to both domestic and commercial users.

A good water supply is also of great value to the manufacturing interests of a town. Many establishments, such as sugar refineries, starch factories, cleaning and dyeing houses, chemical works, etc., require an abundant water supply, and in some cases water of a high degree of purity. The question of water supply indeed often determines the location of factories. Large quantities are also used for operating elevators, for boiler purposes, and for many other uses that may be classed as commercial.

The most important public use of water supply is in extinguishing fires. The economic value of a good fire-protection system is directly shown in the reduced rates of insurance which follow its introduction or improvement. Instead of distributing a heavy fire loss among the people of a community through high rates of insurance it is assuredly much better economy to contribute to the maintenance of a public waterworks, which at the same time provides a suitable water for other purposes. To permit of the establishment of a certain class of factories it is absolutely essential that an efficient fire protection be furnished.

Other important public uses of a water supply are in street sprinkling and sewer flushing, in furnishing water for public buildings, and for drinking and ornamental fountains. A real value exists in the improved appearance which may be given a city by the use of water in fountains and for lawns and public parks; and, indeed, all the benefits accruing from a good water supply act indirectly to increase the desirability of a town for many purposes and to enhance the value of the property therein.

CONSUMPTION OF WATER.

3. General Considerations. When a new or enlarged water supply is under consideration one of the first questions to be answered is that relating to the quantity of water which will be required in the near future. The knowledge which is required includes not only the average daily quantity which will be needed, but also the monthly, daily, and hourly variation in the rate of consumption. In designing certain parts of the works the average consumption for the year is sufficient, but in certain other parts, such as pumps and distributing pipes, we need to know the greatest rate of consumption for a very short period of time.

There are many influences which affect the rate of consumption per capita of any given town or city. One of these is the actual population of the town. Thus in large cities the use of the public supply is almost a necessity, while in small towns and villages the private supplies may remain in use to a large extent long after the introduction of the public water supply.

The nature of the industries of a town is a large factor in determining the amount of water used; also the wealth and habits of the people, and the extent to which water is used for fountains, watering of lawns, street sprinkling, and other public purposes. Climate has also a very considerable influence, especially as to the amount used for sprinkling purposes and that which is wasted in winter to prevent freezing. It is probable, however, that the most important factors in determining the consumption is the degree of care taken to detect leakage or waste, and the fact as to whether the water is sold by measure or otherwise. Good quality, abundant quantity, and high pressure tend to increase the consumption by encouraging a more liberal use and often, at the same time, greater wastefulness.

4. The Average Daily Consumption Per Capita. In Table No. 1 are given the rates of consumption per capita for several American cities and towns in 1895.

It will be noted from Table No. 1 that a great variation exists in the rate of consumption in different cities and that the consumption in some of the cities is very high. For example, it is 271 gallons in Buffalo, New York, and 247 gallons in Allegheny, Pennsylvania. It will also be noted from a comparison of Tables No. 1 and 2 that the consumption is, on the average, much less in European than in

TABLE I.

Consumption of Water in American Cities and Towns.

City.	Population. 1900.	Daily consumption per inhabitant. 1895.
New York	3,437,202	100
Chicago	1,698,575	139
Philadelphia	1,293,697	162
Brooklyn		89
St. Louis	575,238	98
Boston	560,892	100
Cincinnati	325,902	135
San Francisco	342,782	63
Cleveland	381,768	142
Buffalo	352,387	271
New Orleans	287,104	35
Washington	278,718	200
Montreal		83
Detroit	285,704	152
Milwaukee	285,315	101
Toronto		100
Minneapolis	202,718	88
Louisville	204,731	97
Rochester	162,608	71
St. Paul	163,065	60
Providence	175,597	57
Indianapolis	169,164	74
Allegheny	129,896	247
Columbus	125,560	127
Worcester	118,421	66
Toledo	131,822	70
Lowell	94,969	82
Nashville	80,865	139
Fall River	104,863	35
Atlanta	89,872	42
Memphis	102,195	100

In Table No. 2 are given the rates of consumption for several European cities.

TABLE 2.

Consumption of Water in European Cities.

City.	Estimated population.	Daily consumption per capita, gallons.
London	5,700,000	42
Manchester	849,093	40
Liverpool	790,000	34
Birmingham	680,140	28
Bradford	436,260	31
Leeds	420,000	43
Sheffield	415,000	21
Berlin	1,427,200	18
Breslau	330,000	20
Cologne	281,700	34
Dresden	276,500	21
Paris	2,500,000	53
Marseilles	406,919	202
Lyons	401,930	31
Naples	481,500	53
Rome	437,419	264
Florence	192,000	21
Venice	130,000	11
Zurich	80,000	60

American cities. Both of these variations are due largely to the variation in practice in the use of meters to measure the water used and to charge accordingly. In some American cities meters are quite generally used, and without exception the consumption of water in those places is comparatively low. Meters are also generally used in European cities with the results as indicated in the table. It is true, however, that there is a greater general use of water for proper purposes in this country than in foreign countries.

5. Consumption of Water for Different Purposes. In studying the subject of the consumption of water it is desirable to consider the different uses of water under the following heads: (1) Domestic use; (2) Commercial use; (3) Public use; (4) Loss and waste.

(1) *Domestic Use.* Statistics collected from many sources where the supply has been actually measured by meter show that the amount of water used for domestic purposes will vary from about 15 to 40 gallons per capita; usually from 20 to 30 gallons. Where the supply is not metered, but is paid for according to the number and kind of fixtures in use, or the number of rooms in the house, the consumption may be several times the above figures. It has been known in some cases to go as high as 175 and 200 gallons per capita. Under these conditions it is difficult to predict what the consumption will be.

(2) *Commercial Use.* Under this head are included all uses for mechanical, trade, and manufacturing purposes. Large users of water for such purposes are office buildings and stores, hotels, factories, elevators, railroads, breweries, sugar refineries, and a few other industries. In large cities the use for commercial purposes is likely to be more than in small cities. Various statistics show a consumption for these purposes of 10 to 40 gallons per capita. The nature of the industries will determine very largely this item.

(3) *Public Use.* This includes the water used for schools and other public buildings, street sprinkling, water troughs and fountains, sewer flushing and the flushing of water mains, fire extinguishment, and a few other occasional uses. Water for such purposes is seldom measured, but the amount is not likely to exceed on the average a few gallons per capita, although the rate of consumption is far from being uniform. The water used for street-sprinkling purposes is likely to be quite a large proportion of the total, as much as 10 gal-

lons per capita being used in some places. The average is, however, not more than one or two gallons per capita. For fire purposes the total consumption is relatively small, but during fires the rate of consumption is very high for a short time. The total consumption for public purposes may be estimated from 3 to 10 gallons per capita.

(4) *Loss of Water.* The chief cause of waste is bad plumbing and carelessness on the part of the private consumer, but this source of waste has already been mentioned under the first item. There is in addition considerable waste due to leakage of mains and reservoirs and minor uses of water, not included under the foregoing. It is estimated that at least 15 gallons per capita should be allowed for this item.

From the foregoing analysis it may be concluded that a reasonable estimate of the consumption of water where meters are largely used will be about 40 gallons as a minimum and 120 gallons as a maximum; 75 or 80 gallons may be taken as a fair allowance under average conditions. Where meters are not used extensively the statistics in Table No. 1 show that 200 gallons per capita would not be an excessive figure, but it is impossible under such circumstances to make a very close estimate.

6. Variations in Consumption. The foregoing sections have discussed only the average consumption throughout the year. There will now be considered the variations which occur in the consumption from time to time.

Monthly Variations. In nearly all cases the rate of consumption reaches a maximum in the summer owing to the use of water for street and lawn sprinkling. This high rate usually extends over two or three months. A secondary maximum often occurs in the winter, due to the waste of water to prevent freezing, but the use of meters will largely prevent excessive variations from this cause. In extreme cases, however, the winter consumption may be very high. The monthly variations in consumption for several places are illustrated by the data given in Table No. 3.

From the table it may be concluded that the maximum monthly rate will seldom exceed 125 per cent of the average, it being in fact much below this figure for most places represented. Excessive consumption is likely to continue for two or three consecutive months, averaging for this longer period a rate of 110 to 115 per cent of the yearly average.

Daily Variations.—The maximum daily rate is usually estimated at about 150 per cent of the average. In Table No. 3 very considerable differences are to be noted in the ratios for different places, these being caused by a variety of conditions, some accidental and some constant.

TABLE 3.

Maximum Monthly and Daily Ratios Expressed as Percentages of Average Consumption.

City.	Ratio of maximum monthly to average consumption.	Ratio of maximum daily to average consumption.	City.	Ratio of maximum monthly to average consumption.	Ratio of maximum daily to average consumption.
Chicago	108	116	Louisville	127	135
Philadelphia	110	122	Columbus	107	157
Boston	114	119	Fall River	115	......
Cincinnati	124	153	Dayton	118	178
Cleveland	114	146	Newton	125	143
Buffalo	......	168	Pawtucket	111	153
Detroit	117	150	Woonsocket, R. I.	122	155
Milwaukee	113	......	Marquette, Mich.	139	194

The maximum daily rate will usually occur in the month of maximum consumption, and a rate considerably above the average for the month will occur for several consecutive days. Thus where the maximum daily consumption is 150 per cent of the average, the maximum weekly consumption is likely to be from 130 per cent to 140 per cent of the average, but for longer periods of time the rate will approach the monthly maximum.

Ordinary Hourly Variations. If there were no waste or leakage, the consumption during several hours of the night would be almost nothing and the consumption during several hours of the day would be two or three times the average for the twenty-four hours. It is a fact, however, that the rate of consumption at night is usually as much as 60 per cent of the average, and during the hours of maximum consumption it is not often more than one and a half times the average. Where waste is carefully prevented, and the consumption therefore low, the variation during the twenty-four hours will be relatively greater than where the waste is great and the total consumption great. Fig. 1 shows typical curves representing the hourly variation in consumption throughout the day. The curve for New York illustrates what occurs in a city where waste is fairly large, while that for Des Moines represents a case where consumption is small and the waste largely prevented. The average daily consumption per capita for New York was 100 gallons and for Des Moines 43 gallons.

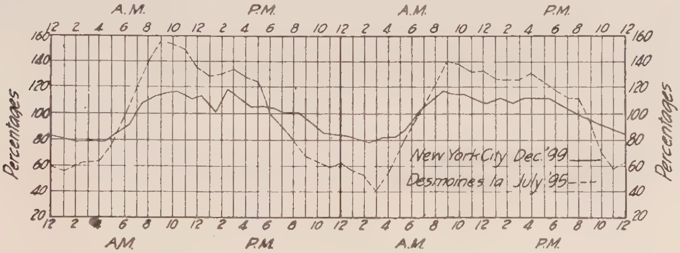


Fig. 1. Typical Curves Showing Hourly Variations of Water Consumption.

Consumption for Large Fires. The consumption for large fires must be considered in addition to the rates given above, The maximum rate of fire consumption in gallons per capita per day for a town or city of average character may be taken equal to $\frac{1000}{\sqrt{x}}$, where x = population in thousands. This is based on Mr. Kuichling's estimate of the required number of fire streams.

If, for example, the average consumption is 100 gallons per capita, then the fire rate in per cent of the average will be as follows for different size cities:

Population.	Rate of fire consumption in percentage of average, when average equals 100 gallons per.day.
1,000	1000 per cent.
5,000	447 "
10,000	316 "
50,000	141 "
100,000	100 "
200,000	71 "
300,000	58 "
500,000	45 "

For other values of the daily consumption the percentages would vary accordingly, being greater for smaller consumptions. In the case of small cities the fire rate is evidently the principal factor to be considered; in large cities it is of much less relative importance. The duration of the above rate of fire consumption may be several hours; it has been estimated by Freeman at about six hours as a maximum.

The Combined Maximum Hourly Rate. In obtaining the total maximum rate of consumption at times of fires it is not necessary to assume that a great fire will occur coincident with the maximum use for other purposes. In fact, at times of great fires the use of water for many purposes would be interrupted. For average conditions the following may be taken as a reasonable allowance:

If the average daily rate is 100 per cent, then the maximum daily rate equals 150 per cent, and adding 20 per cent for increased consumption during day gives a total of 180 per cent. To this the fire consumption should be added by the use of the formula of the preceding paragraph.

Example. If the average daily consumption of a city of 20,000 inhabitants equals 80 gallons per capita, what will be the approximate maximum rate of consumption (a) for the day of greatest consumption, and (b), at the occurrence of a large fire?

From the foregoing discussion the maximum daily consumption may be estimated at 80×150 per cent $= 120$ gallons per capita.

From the above estimate the rate for ordinary use may be taken at 180 per cent of 80 gallons or $80 \times 180 = 144$ gallons. The fire rate $= \frac{1000}{\sqrt{20}} = 224$ gallons per capita. The total rate is therefore $144 + 224 = 368$ gallons per capita per 24 hours.

7. Growth of Cities. A necessary factor in any estimate of future consumption is that of future population. The rate of growth of different cities is exceedingly various, but of any one city it is likely to be fairly constant for several years, or at least will vary but slowly. The older and the larger the city the more uniform the rate of growth, and, barring national disasters, a fairly close estimate can be made for two or three decades in the future.

Probably the best way to estimate the population of a city for several years in the future is to take as a basis its growth in past years. If conditions do not change the per cent added each year or decade is apt to remain about the same. No one can predict closely the growth of a city, and in water-supply problems a close estimate is unnecessary.

Example. If the population of a city be 5,250 in 1880, 7,670 in 1890 and 11,400 in 1900, estimate the population in 1910.

The growth from 1880 to 1890 was 2,420, equal to 46 per cent, and from 1890 to 1900 it was 3,730, equal to about 49 per cent of the population in 1890. These figures show a steady growth, and it may be assumed that for the next decade the growth will be about the same, say 48 per cent. Then, 48 per cent of 11,400 = 5,472, and the estimated population in 1910 = 11,400 + 5,472 = 16,872.

The population for 1920 may be estimated in the same way, but the result will be much more uncertain than for 1910.

SOURCES OF SUPPLY.

8. Classification. The sources of water supply may be divided into the following classes, according to the general source and the method of collection:

A. Surface waters:
   1. Rain water collected from roofs, etc.
   2. Water from rivers.
   3. Water from natural lakes.
   4. Water collected in impounding reservoirs.

B. Ground waters:
   5. Water from springs.
   6. Water from shallow wells.
   7. Water from deep and artesian wells.
   8. Water from horizontal galleries.

Each of the above sources except the first and last is at present furnishing many cities in the United States with a more or less satisfactory water.

The following table gives the number of waterworks in 1896 obtaining their supply from the various sources indicated.

Region.	Surface waters.	Ground waters.	Total.
Northeastern States	615	511	1,238
Southeastern States	143	180	340
North Central States	193	469	715
Western States	329	662	1,063
Total	1,280	1,822	3,356

SURFACE WATER SUPPLIES.

9. Rainfall is the source of all water supply, whether it be caught as it flows over the surface or is first allowed to percolate into the ground to furnish water for wells and springs. The amount of rainfall is expressed in inches of depth upon a horizontal surface, snowfall being reduced to its equivalent amount of rainfall. With the ordinary rain gauge it is impracticable to determine rates of rainfall for short periods of time, the records usually obtained from these gauges being merely the total amounts of rainfall for each twenty-four hours. For estimating flood volumes from small areas, however, it is important to know the rate of rainfall for much shorter periods than one day. For this purpose self-recording gauges are essential, that is, gauges which give a continuous record of the rainfall or a record taken at such short intervals as to be for all practical purposes continuous. Various forms have been devised, some weighing the water, others recording by volume.

Rainfall statistics for a large number of stations can now be readily obtained from the monthly reports of the Weather Bureau. The data of importance in connection with water-supply questions are the mean yearly rainfall, the deviation from this in dry years, the monthly rainfall, and finally the maximum depth of rain falling in a single day or less.

10. Mean Annual Rainfall. The mean annual rainfall for a number of stations in the United States is shown in Table No. 4. The table also gives the ratio of the rainfall in the driest year, covered by the statistics, to the average.

The maximum rainfall is along a narrow belt of the North Pacific coast, where it considerably exceeds 60 inches. Towards the interior the amount rapidly falls off, and between the Sierras and the Rocky Mountains it ranges from 5 to 15 inches. East of the Rockies there is a gradual increase eastward and southward to a maximum along the Gulf of 60 inches, and from 40 to 50 inches on the Atlantic coast. The table also shows that in the driest years the rainfall is in most places only 50 to 60 per cent of the average. In the central and Western States the variation is greater than in the Eastern States.

The monthly distribution of the rainfall is of great importance in all questions relating to the utilization of water for power purposes or for the supply of cities. The rain falling in the summer months, when vegetation is using a maximum of water and evaporation is rapid, is of but little value for supplying water to the streams. It is the winter and spring rains which must largely be relied upon to fill reservoirs and to raise the low ground water to its normal level.

TABLE 4.

General Rainfall Statistics for the United States.

Station.	Mean yearly rainfall, Inches.	Per cent rainfall, driest year to mean rainfall.
Boston	45.4	60
New York	44.7	62
Philadelphia	42.3	70
Charleston	49.1	48
Jacksonville	54.1	74
Shreveport	48.2	67
Mobile	62.6	68
New Orleans	60.3	64
Vicksburg	52.7	70
Louisville	47.2	74
Cairo	42.6	62
Cincinnati	42.1	60
Cleveland	36.6	71
Marquette	32.3	69
Chicago	34.0	66
Milwaukee	31.0	66
St. Louis	40.8	55
St. Paul	28.2	53
Duluth	30.7	65
Omaha	31.4	57
North Platte	18.1	56
Denver	14.3	59
Salt Lake City	18.8	55
Spokane	18.6	73
Santa Fe	14.6	53
Yuma	2.8	25
San Diego	9.7	30
Los Angeles	17.2	33
San Francisco	23.4	51
Portland	46.2	67

11. Maximum Rates of Rainfall. In estimating the maximum flood discharges of small streams—a matter of very great importance in the design of dams and reservoir embankments—it is desirable to know the maximum rates of rainfall for periods of a few hours or a single day. Great rainstorms occur but rarely, but in hydraulic works where a failure would mean not only the destruction of property but often a great loss of life, it is necessary to provide against the greatest flood ever likely to occur. Accurate data of such floods must be based on many years of observation, but extraordinary rainfalls are likely to occur almost anywhere, and it may be assumed that what has happened in one locality may happen at any place in the same region. Examination of the data contained in the United States Weather Bureau Reports shows that in the Northern and Central States a rainfall at the rate of 4 inches for one hour and 8 inches for 24 hours represents the greatest rain likely to occur; in the South Atlantic and Gulf States these figures should be about 4 inches for one hour and 10 inches for 24 hours.

That excessive rainfalls are of sufficient extent to cover areas of such size as are ordinarily considered in water-supply problems is shown by the statistics of great storms. In October, 1869, a great storm occurred in the eastern part of the United States, with its maximum intensity in Connecticut. A careful analysis of the records made by Mr. James B. Francis shows the areas covered by different depths of rain to have been as follows:

Depth of rain.	Area covered.
6 inches or more	24,431 square miles.
7 " " "	9,602 " "
8 " " "	1,824 " "
9 " " "	1,046 " "
10 " " "	519 " "
11 " " "	179 " "

The following are some of the maximum rates observed in this storm:

4.00	inches in	2	hours.
4.27	"	3	"
5.86	"	18.5	"
7.15	"	24	"
8.90	"	30	"
8.44	"	42	"

FLOW OF STREAMS.

12. When a stream is under consideration as a source of water supply, the peculiarities of its flow—the minimum, maximum, and total flow for various periods of time—are among the first things to be determined. The most accurate as well as the most direct method of determining these is by means of a series of gaugings extending over several years, but, where gaugings are not to be had, or where they are very limited in extent, as close an estimate as possible must be made from a comparison with other streams whose flows are known, taking into account as far as may be the differences in rainfall, climate, and in the various characteristics of the different watersheds.

Rainfall is expressed in inches in depth, and the rate in inches per hour or per twenty-four hours; and for comparative purposes stream flow is often likewise expressed, meaning thereby inches in depth over the entire watershed. For other purposes the flow is usually expressed in cubic feet, or cubic feet per square mile of watershed, and the rate of flow in cubic feet per second, or cubic feet per second per square mile. The foot and second units are also convenient to use in all hydraulic formulas, but in matters pertaining to storage and distribution the gallon unit is in common use, and rates are expressed in gallons per minute and gallons per twenty-four hours.

For convenience in computations relative to rainfall and flow of streams, the following table is inserted:

TABLE 5.

Volumes and Rates of Flow in Feet and Seconds Corresponding to Given Volumes and Rates of Rainfall in Inches and Hours.

Depth in inches.	Cubic feet per square mile.	Inches per hour.	Cubic feet per second per square mile.	Inches per 24 hours.	Cubic feet per second per square mile.
0.1	232,320	0.1	64.5	1	26.9
0.2	464,640	0.2	129.0	2	53.8
0.3	696,960	0.3	193.5	3	80.7
0.4	929,280	0.4	258.1	4	107.5
0.5	1,161,600	0.5	322.6	5	134.4
0.6	1,393,920	0.6	387.1	6	161.3
0.7	1,626,240	0.7	451.7	7	188.2
0.8	1,858,560	0.8	516.2	8	215.1
0.9	2,090,880	0.9	580.7	9	242.0
1.0	2,323,200	1.0	645.3	10	268.9

One inch of rain = 2,323,200 cu. ft. per sq. mile.
One inch per hour = 645.33 cu. ft. per sec. per sq. mile.
One inch per 24 hours = 26.89 cu. ft. per sec. per sq. mile.
One cubic foot = 7.4805 U. S. gallons.
One cubic foot per sec. = 646,300 gallons per day.

The question of the flow of streams naturally divides itself into three parts:

First, the minimum flow of the stream.

Second, the maximum or flood flow.

Third, variations in the flow through successive months and years.

The first information is necessary in case a stream is under consideration for which but little storage is obtainable, or in answer to the question whether it is practicable to draw directly from the stream without storage. The second is of great importance in the design and execution of all river work, and especially in determining the size of waste weirs. The third determines the supplying capacity of the watershed and the size of impounding reservoirs.

EXAMPLES FOR PRACTICE.

1. If a rain is falling at the rate of $\frac{1}{2}$ inch per hour, how many cu. ft. per sec. will this amount to over an area of 10 sq. mi.? 3,226 cu. ft. per sec. Ans.

2. If 1 inch of water is collected from an area of 20 sq. mi., how many days will this supply a town of 15,000 inhabitants using 100 gallons per capita daily?

The total amount of water collected = 2,323,200 × 20 = 46,464,000 cu. ft. = 347,500,000 gal. This will last 231 days. Ans.

13. The Dry-Weather Flow. The dry-weather flow of streams is maintained entirely from ground and surface storage; and as facilities for such storage vary in different watersheds, so will the minimum flow vary.

In Table No. 6 are given the minimum flows of several streams in different localities. It will be seen that the minimum varies greatly with the size of the stream and locality, and that streams of several hundred square miles of drainage area may have a minimum of zero.

TABLE 6.

Minimum and Maximum Flow of Streams.

Stream.	Place.	Drainage area. square miles.	Minimum flow. cubic feet per sec. per sq. mile.	Maxim'm flow. cubic feet per sec. per sq. mile.
	New England.			
Merrimack........	Lawrence.......	4,599	0.31	20.87
Connecticut.......	Hartford	10,234	0.51	20.27
Nashua	Massachusetts...	109		104.5
Sudbury..........	Massachusetts...	78	0.036	44.2
	New York.			
Chemung	Elmira	2,055		67.1
Croton West Br....	....	20.37	0.016	54.43
	New Jersey.			
Delaware.........	Stockton	6,790	0.17	37.5
Pequannock	....	48		115
	Pennsylvania.			34.9
Perkiomen	Frederick.......	152	0.39	
South Fork	Dam in Creole Township	48.6		215
	Maryland.			131
Potomac	Cumberland	1,364	0.018	
	Illinois.			
Rock	Rockford	6,500	0.0158	
Des Plaines	Riverside	630	0	21.4

14. Flood Flow. The maximum rate at which the waters from great storms will pass down a stream is affected largely by the steepness of the slopes, by the size and shape of the drainage area, and by the distribution of the branches. Small areas will have larger maximum rates of flow than large areas, other things being equal, as the former are affected by short rainfalls of high rates, while in the latter case the maximum flows are caused by rains of longer duration but of less intensity. For a like reason streams with steep slopes will have a higher maximum rate than those with flat slopes.

Of great importance in distributing the run-off over a long interval of time, and so reducing the maximum rate, is the surface storage of natural lakes and ponds and of those created by the inundation of large flats bordering the stream. The effect of this last factor may be sufficient to reduce the flood flow to one-half or one-fourth that of a stream with a narrow valley.

In Table No. 6 great variation in the maximum flow is observable, due partly to the varying rates of rainfall, but largely to the different characteristics of the streams. Various formulas have been proposed for expressing the maximum flow of a stream, some involving only the rainfall and area, while others attempt to take account also of the slope and shape of the watershed.

Among the most widely known of this class of formulas is that given by Fanning and recommended by him as applicable to average New England and Middle-State basins. It is

$$Q = \frac{200}{\sqrt[6]{M}} \qquad (1)$$

in which Q = discharge in cubic feet per second per square mile and M = area in square miles. It gives results probably somewhat too low for small areas.

Example. What will be the flood flow according to formula 1 for a drainage area of 10 square miles?

The flow will equal $\frac{200}{\sqrt[6]{10}} = 136$ cu. ft. per sec. per sq. mi.

TABLE 7.

Statistics of the Yearly Flow of Streams.

Stream.	Area drained, square miles.	Average yearly flow.		Dry year flow.	
		Rain, inches.	Flow, per cent of rainfall.	Rain, inches.	Flow, per cent of rainfall.
Cochituate	18.87	47.08	43.2	31.20	31.3
Croton	338.0	48.38	50.8	38.52	37.8
Genesee	1,060	39.82	32.5	31.00	21.5
Perkiomen	152	47.98	49.2	38.67	40.4
Potomac	11,043	45.47	52.7	37.03	39.2
Savannah	7,294	45.41	48.9	43.10	37.7
Upper Mississippi	3,265	26.57	18.4	22.86	7.1

15. Annual Discharge. Table 7 gives some statistics of the annual flow of streams as compared to rainfall. It will be seen that in the dry years the percentage running off is much less than in the average year. From these data and other statistics it is estimated that for a stream of average conditions east of the Missouri and Mississippi Rivers the percentage of rainfall flowing off for different annual rainfalls is about as follows:

Rainfall, inches.	Per cent running off.
20	25 to 35
30	30 to 40
40	35 to 45
50	40 to 50

In the nature of the problem there is a wide variation in percentage due to variations in the conditions of the watershed, climate, etc. Whatever tends to promote evaporation from the watershed decreases the run-off. Thus a watershed with a large percentage in grass will yield a less amount than one with rocky and barrėn hillsides; one with a large percentage of water surface, less than one with a small percentage. Again, the higher the temperature the greater the evaporation and the less the stream flow. Steep, rocky hillsides will give a large per cent of the rainfall to the streams, but the flow will be very irregular; flat grass lands will give little or nothing to the streams during the season of growth. All these things must be considered in estimating the flow of a stream from rainfall data and from statistics of the flow of other streams.

Examples. Estimate the flow of a stream during dry years where the average annual rainfall is known to be 40 inches.

The rainfall for a very dry year may be taken from Table No. 4 at say 60 per cent of the average or 40 × .60 = 24 in. For a rainfall of this amount the per cent running off will probably be between 25 and 35. If this is an average watershed we may put it at about 32 per cent. The run-off will then be estimated at 24 × .32 = 7.7 inches.

Note. The wide variation in percentage indicates that such estimates as this are very uncertain. Actual stream measurements are the only safe guide.

2. How much water can probably be collected in a dry year from an average watershed where the rainfall in very dry years is 30 inches?

By the estimates of section 15 it is probable that at least 33 per cent will run off or can be caught in a reservoir. This amounts to 30 × .33 = 9.9 inches.

By Table No. 5 this amounts in gallons per sq. mi. to 9.9 × 2,323,200 × 7.48 = 172,000,000 gallons.

16. Monthly Variation in Stream Flow. During dry years very little water can be collected from summer rains. Dependence must be had on winter snows and spring rains for filling storage reservoirs and nearly all the yearly supply will be caught in the months from December to May inclusive. During average seasons a large proportion of the stream flow occurs in the summer months. Generally about three-fourths of the yearly flow occurs in the months from December to May and only one-fourth from June to November, whereas in very dry years the summer flow may be considered as practically nothing.

17. Quality of Surface Waters. Surface water supplies are drawn from two general sources—rivers and lakes. River supplies may be divided into those obtained directly from large rivers and those obtained from impounding the flow of small streams in reservoirs. The quality of surface waters may be considered with reference to: (1) appearance, (2) mineral content, (3) the presence of disease-producing organisms.

(1) The appearance of a water is affected by the presence of clay and sand in suspension, rendering the water turbid, and by certain vegetable material giving the water a distinct color. Turbidity varies according to the nature of a watershed. While a turbid water is very objectionable for household use it cannot be said to be actually dangerous. Turbidity is removed by allowing the water to rest in

reservoirs, thus permitting the clay to settle, or by passing the water through filters. Surface waters flowing through swampy regions are usually colored, due mainly to the extraction of soluble coloring matter from vegetable material. Such peaty waters, while perhaps unsightly in appearance, may be, however, perfectly wholesome in spite of this physical defect.

(2) While flowing surface waters do not dissolve so much mineral matter as ground waters, yet they take up an appreciable amount, depending considerably on the character of the soil over which they pass. A large part of the mineral content is usually carbonate of lime. In general surface waters are preferable to ground waters as regards their mineral content, a hard water (one containing lime) being less desirable for culinary and manufacturing purposes.

(3) The most important question relating to the quality of a water is whether it is dangerous to the health. It has been well demonstrated that certain diseases, particularly cholera and typhoid, are caused by certain minute organisms called bacteria. These inhabit the intestinal tract of persons sick with the disease and are present in enormous numbers in the sewage wherever these diseases exist. Whenever such sewage or drainage gets into the water supply of any town an outbreak of the same epidemic is sure to appear. Many cases are on record of whole villages being affected through the contamination of the water supply by a single diseased person. From such facts it is seen that the quality of a water supply from this point of view is exceedingly important.

A surface water supply can be absolutely safe only when it is drawn from an uninhabited area. A few scattered farm houses, if not located too near a water course, are not likely to cause serious pollution. But where the watershed is quite populous, and especially where villages are located in the valleys, the danger of the transmission of disease through the water supply is very great.

The danger in the use of water from a large stream depends on the amount and nearness of the pollution. All large streams receive more or less drainage from towns and cities, but if such pollution is relatively small and remote the danger is small. As a rule a surface water supply is not free from danger unless the water is artificially purified by some adequate means, but many large cities in the United States continue to use water supplies which are badly contaminated.

The result of such use is shown in the relatively high death rate from typhoid fever in such places.

The quality of lake supplies is likely to be better than that of rivers. Such water is usually quite free from turbidity, as the sediment brought into it by the tributary streams soon settles; and unless polluted by sewage in the immediate neighborhood, it is likely to be relatively safe from a sanitary point of view. Experiments show that in the settling of the clay and sand particles, the bacteria settle to a great extent, and a marked purification takes place in a polluted water. For the same reason that lake water is better than river water, it is true that a supply from a small stream is usually improved by storage in a large storage reservoir. Sometimes, however, vegetable growths occur in reservoirs which give to the water a disagreeable odor.

GROUND WATER SUPPLIES.

18. Occurrence of Ground Water. The rain which falls upon the ground is disposed of in three ways: A part flows off immediately in the streams, a part is evaporated from the ground and vegetation, and a part percolates into the soil.

Percolating water that escapes beyond the reach of vegetation must, in obedience to the law of gravitation, pass on downward until it reaches an impervious layer of some sort. The immediate impervious stratum is the surface of the water which has preceded it and which has in past ages filled every pore and crevice of the earth's crust up to a certain level at which the escape of the water laterally becomes equal to the addition from percolation. The accumulation of water which thus exists in the ground is called *ground water*, and its surface the *ground-water level* or the *water table*.

In limestone regions it is sometimes the case that quite large streams are found flowing underground, and large cavernous spaces may be converted into underground lakes of considerable size, as in the great caverns of Indiana and Kentucky. Such bodies of water are, however, rarely available for a water supply, and it may be taken as a safe rule for ground-water supplies dependence must be placed upon the water which percolates into and flows through the pore-spaces in soils and rocks, the amount of which is strictly dependent upon the rainfall and the laws of hydraulics that govern the flow.

19. General Form of the Water Table. Under the action of gravity the surface of the ground water always tends to become a level surface, and as long as a supply is maintained through percolation there will be a continual downward and lateral flow which will on the average be equal to the percolation. In surface streams a very light inclination is sufficient to cause a rapid movement of water, but in the ground the resistance to movement is so great that a steep gradient is necessary to maintain even a very low velocity.

If we imagine the ground to be throughout of uniform porosity, the ground-water surface will conform in general outline to the ground surface, but with less variations. Such an ideal condition is represented in Fig. 2. At the margin of streams as at A and B the level of ground and surface waters will coincide. Passing back from the stream the ground-water level will gradually rise, but at a less rate than the ground surface, then descend again into another depression, etc. In the valley there is also a fall parallel to the stream, corresponding to that of the surface water, and the direction of flow will be towards and slightly down the stream in the line of greatest slope.

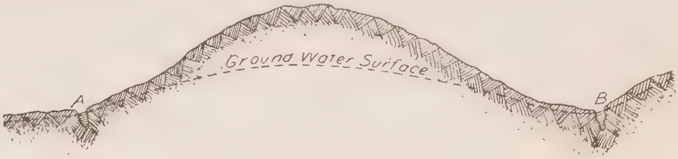


Fig. 2. Relation of Ground Water to Surface Water.

Variations in ground-water level take place comparatively slowly, following gradually the variations in yearly, seasonal, and briefer periods of rainfall. Near streams and in lowlands the level varies little, being fixed largely by the level of the adjacent surface water. At higher points in the water table the level is subject to correspondingly great fluctuations, often many feet in extent. In porous material where slopes are small the variations are small.

20. Porosity of Soils. All soils and rocks near the surface of the earth are capable of absorbing more or less water. In sand of a fairly uniform size the porous space is commonly from 35 to 40 per cent of the entire volume. Mixed sand and gravel will have a smaller percentage of voids, the decrease depending on the variation in size of particles, but it will seldom be less than 25 per cent. Rocks will vary in porosity from a very small fraction of 1 per cent in the case of some granites to 25 or even 30 per cent for some loose textured sandstones.

The amount of moisture which a soil or rock will absorb is, however, not of so much importance to the water-works engineer as is the carrying capacity and the amount which can readily be drawn from such material when previously saturated. In fine soils the movement of the water is so slow and such a large part of the water is retained by capillary action that such soils are of little value as carriers of water; and to obtain economically the large quantities required for public supplies it is necessary that the water-bearing material be of a very open, porous character. Adequate supplies are rarely obtained from anything but sand and gravel deposits, or from very porous rock. The most favorable formations for furnishing large quantities of water, are the various sandstones, conglomerates, and gravel deposits. Sandstones are found which vary in texture from a very compact rock having a very small degree of porosity to a material almost as porous as sand. Uncemented sands and gravels are of course the most favorable as regards porosity, but they are apt to be rather limited in extent.

21. The Flow of Ground Water. It has been explained in the previous section that the water in the ground has in general a slow rate of flow through the ground. Where a supply of ground water of considerable amount is to be obtained this rate of flow is of much importance. To get water from a ground-water " stream " is exactly similar to the taking of water from a surface stream; in both cases the flow of water in the stream must be at least equal to the proposed draught or the supply will be inadequate. The notion is quite common that in many places the water in the ground is inexhaustible. This is an entirely mistaken idea as is well illustrated by the gradual failure of many ground-water supplies.

Ground water in large quantities is usually obtained either from large gravel deposits of comparatively small depth, forming broad underground streams, or from extensive deposits of porous rock like sandstone, the latter source being tapped by deep wells many of which are the well known "artesian" wells. In the case of a gravel deposit near the surface it is often possible to estimate the quantity of water actually flowing through a given section of the deposit.

The best method of estimating capacity of a ground-water source is by means of actual pumping tests carried on for a sufficient length of time to bring about an approximate state of equilibrium between the supply and the demand which will be shown when the level of the water in the trial well ceases to lower. It will rarely be practicable to continue such tests until perfect equilibrium is reached, for in many cases several years of operation would be required to determine the ultimate capacity of a source. Pumping tests of short duration are apt to be very deceptive, as the ground water may exist in the form of a large basin or reservoir with very little movement, corresponding to a surface pond with small watershed, and brief tests would give but little more information than similar tests on a pond.

Where it can be done it is very desirable to get an approximate idea of the amount of water actually flowing per unit of time through the area in question.

To do this we must estimate the velocity of flow, the cross-section of the porous stratum containing the water, and the percentage of porous space.

The rate of flow of ground water streams is very small compared to that of surface streams. It depends on the slope or inclination of the ground and upon the size of the grains of sand or gravel through which it passes. The following table shows about what the velocities are likely to be for various slopes and conditions of soil.

TABLE 8.

Velocities of Flow of Ground Water in Feet Per Day.

Material.	Slope of ground, feet per mile.					
	10	20	30	40	50	100
Fine Sand	0.2	0.4	0.6	0.8	1.0	2.0
Medium Sand	1.5	3.0	4.5	6.0	7.5	15.
Coarse Sand	4.0	8.0	12.0	16.0	20.	40.
Fine Gravel, free from sand ...	20–40	40–80	60–120	80–160	100–200	200–400

The velocity of flow having been determined it remains to estimate the actual quantity of water flowing through a given territory. Of the total volume of a body of sand or gravel, the water will occupy only about 25 to 30 per cent. The actual volume of water, therefore, which will pass through a given section will be only 25 or 30 per cent of the amount were it solid water. If v = velocity of flow in feet per day, A = area of cross-section of the porous bed at right angles to the direction of flow, then assuming a porosity of 25 per cent or $\frac{1}{4}$th, the actual volume of flow per day will be in cubic feet

$$Q = \tfrac{1}{4} vA \qquad (2)$$

Thus suppose we have a porous bed of coarse sand in which the water is 10 feet deep, the bed is 500 feet wide and slopes 20 feet per mile. The velocity of flow by Table 8 may be taken at about 8 feet per day. The cross-section $A = 10 \times 500 = 5{,}000$ square feet. Hence the volume of flow will be approximately $\frac{1}{4} \times 8 \times 5{,}000 = 10{,}000$ cubic feet per day, or about 75,000 gallons. This being the total rate of flow through the sand it is evidently the greatest amount of water that could be extracted from this sand deposit by means of any system of wells or other devices. To give the above results the bed must be of considerable length and the water in it must be about the same depth throughout and have the same slope as the surface.

SPRINGS.

22. Formation of Springs. Springs are formed where, for any reason, the ground water is caused to overflow upon the surface. The conditions causing their formation are varied and should be carefully studied in connection with the design of collecting-works, as upon them depend largely such questions as the constancy of flow, the possibility of increasing the yield by suitable works, and the probable success of a search for additional springs. According to differences in these conditions springs may be divided into three general classes, each of which will be discussed separately.

First Class. The most important class of springs is that in which the water, in its lateral movement, is brought to the surface at the outcrop of a porous stratum where it is underlain by a relatively impervious one. Fig. 3 represents such conditions, the ground water escaping at the outcrop of the impervious material thus forming a spring. The porous stratum may be sand or gravel, or a porous rock; while the impervious layer is usually clay, or rock of an argillaceous character.

There are many cases of large springs of this class, the supplies for some of the largest cities of Europe being obtained from such sources. The city of Vienna is supplied from springs 60 miles distant that occur at the outcrop of a fractured dolomitic limestone underlaid by slate. The largest spring, the Kaiserbrunnen, has an average flow of about 150 gallons per second, varying from 60 to about 250.

Second Class. Under this class are considered those springs where the water-bearing stratum is covered to a greater or less extent by an impervious one, and which are therefore more or less artesian in character. In this case the water finds its way to the surface where the overlying impervious material is broken, or through a fault, or it breaks through at places where it is not sufficiently strong or compact to resist the upward pressure.

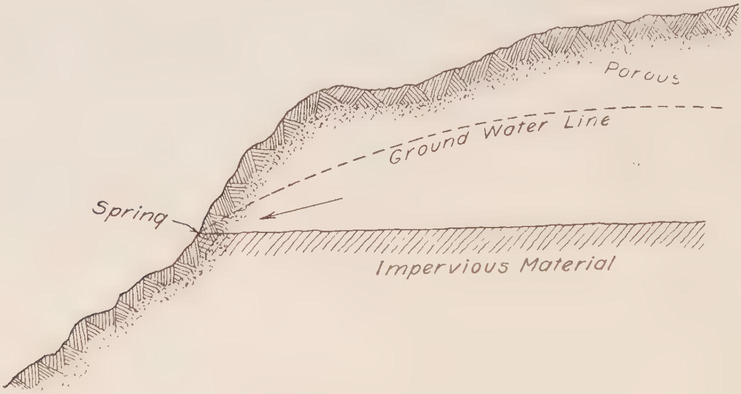


Fig. 3. Formation of Springs.

In some cases springs of this character are fed by water coming long distances through extensive formations which at other points offer conditions favorable for artesian wells. Conditions of this sort give rise to the peculiar phenomenon of large fresh water springs which boil up in the ocean several miles out from the Florida coast, and it is supposed that the great springs in northern Florida are from a similar cause.

Third Class. The third class of springs are mere overflows of the ground water, and occur whenever the carrying capacity of the porous material is insufficient to convey the entire tributary flow. Such conditions also give rise to marshy places at the foot of hills and even on side hills.

23. The Yield of Springs. The yield of any particular spring can readily be determined by weir measurements, and if these are carried out through a period of drought they will give all needed information regarding the supplying capacity of the existing spring.

Springs of the first class will vary in yield with the variations in ground-water level and, therefore, will vary with the rainfall, but will not wholly cease to flow if the water is intercepted by suitable constructions.

Springs of the second class are apt to be much less affected by variations in rainfall than either the first or the third class.

Where a spring of this class exists, investigation may show that the ground-water stream from which it is fed is of considerable size and that the water of the spring is but a small portion of the entire flow. In such a case the yield may be increased by simply enlarging the opening, or by sinking wells and pumping therefrom, as in the case of an ordinary ground-water supply.

Springs of the third class are liable to very great fluctuations, the flow often ceasing entirely.

ARTESIAN WATER.

24. General Conditions. Whenever a water-bearing stratum dips below a relatively impervious one the former becomes in a sense a closed conduit or pipe, and if the flow out of this conduit at the lower end be impeded from any cause, the water will accumulate and exert more or less pressure against the impervious cover. The amount of this pressure will depend on the extent to which the flow is obstructed and on the elevation of the upper end of the conduit, that is, of the outcrop of the porous stratum. If a well be sunk through this impervious stratum at any point, the water will rise in it in accordance with the pressure; and if the surface topography and pressure are favorable, the water may rise to the surface, or considerably above, in which case the well becomes a true artesian, or flowing, well.

Fig. 4 shows an ideal condition for artesian or flowing wells. If A B is a porous stratum outcropping at A and B and covered by an impervious stratum of clay or impervious rock, water entering at A could escape at the lower end B, but at intermediate points would exert a pressure on the covering. If the resistance to flow were uniform, and no water could escape except at B, the decrease of head from A to B would be uniform, or in other words the hydraulic grade line would be a straight line A B. Water would rise to this line in a tube sunk to the porous stratum, and a flowing well would be possible wherever the surface of the ground lies below this line.

Actual conditions may be much modified from those represented in Fig. 4, as where the water is prevented from flowing out at B by reason of an increased density of the stratum or by the stratum becoming thinner. The effect in causing the water to exert an upward pressure is, however, the same. The water-bearing stratum is most often a porous sandstone, although artesian water is also obtained from limestone and in many places from extensive strata of loose uncemented material.

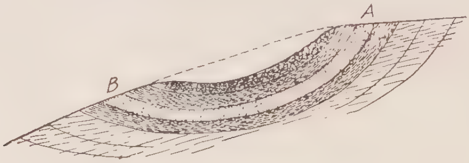


Fig. 4. Dip in Water-Bearing Stratum.

The overlying impervious strata usually consist of clays and shales, these being practically impervious except where fissured. Probably some leakage always takes place through such strata; and many instances are known of large springs which occur at points where the overlying stratum is broken as noted in the preceding section. Except in the case of very limited areas, the capacity of an artesian source as a whole is a question of little importance where it is to be used only for water-supply purposes in towns widely separated; for the total amount of water capable of being drawn from porous rock strata, often hundreds of feet thick and having an outcrop of hundreds or thousands of square miles, is ordinarily very great as compared to any possible demands for such purposes. But wells sunk to tap an artesian stratum must not be placed too close together else they will interfere with one another and the yield per well will be reduced.

25. Predictions Concerning Artesian Wells. The question of the existence of water-bearing strata at any point, their character and depth, and the location of outcrops, is a geological one;

and where full information on this point has not been gained by the sinking of wells or by borings, a geologist familiar with the region in question should be consulted. Much money has often been wasted in fruitless attempts to obtain water in areas and at depths where none could be expected, and frequently such work has been carried on contrary to the advice of experts.

In the construction of wells it is important to preserve samples of the borings, as it is largely through these that a knowledge of the geology of the region is acquired. Chemical analyses of the water are also a valuable aid in identifying strata.

QUALITY OF GROUND WATER SUPPLIES.

26. The quality of ground waters is, in general, quite different from that of surface waters. By percolating through the ground, practically all suspended matter is filtered out, and ground waters are usually clear and sparkling. At the same time this very filtration process causes the water to dissolve more of mineral substances, and the result is that ground waters usually contain much more mineral matter than surface waters. In a limestone country the ground water will be hard, as it will contain lime, and where the soil contains alkali the water will be changed with it. Water that contains little beside lime is not especially objectionable for drinking purposes, but for most other purposes it is more or less expensive and troublesome. An alkali water may be quite unusable.

As regards disease organisms a ground water is likely to be quite free on account of the filtering action of the soil. In the case of private wells, often located near outhouses, pollution is much more likely to occur than in public supplies where any source of pollution must be quite remote.

The temperature, odor and taste of ground waters are generally much more satisfactory than of surface waters. Ground waters constitute a most valuable source of supply for small cities and towns, and where such a supply can be had it should almost always be chosen in preference to a surface water.

CONSTRUCTION OF WORKS.

Before passing on to the details of waterworks construction it will be of assistance to obtain a general view of the subject, and to that end we will here briefly outline the various general features which go to make up a waterworks system.

27. Classification. The various constructive features of a water supply system may be divided into three groups –works for the collection of water; works for the conveyance and distribution of water; works for the purification of water.

28. Works for the Collection of Water. These are divided according to the nature of the source into: (A) Works for taking water from large streams or natural lakes; (B) Works for the collection of ground water; (C) Works for the collection of water from small streams by means of impounding reservoirs.

(A). Works for taking water from large streams or lakes vary in character from a simple cast-iron pipe extending a short distance from shore, to the expensive tunnels and cribs of some of the large cities on the Great Lakes. The location of these works is determined very largely with respect to the quality of the water obtainable. Wherever, as is often the case, it is desired to draw a supply from a lake which at the same time receives sewage from the city, the question is one involving difficulties.

(B). Works for the collection of ground water consist of various forms of shallow wells, artesian wells, filter galleries, etc. The location of works of this class is determined, primarily, by the location of the water-bearing strata. If these are extensive, it will usually be convenient and economical to place the wells at relatively low elevations in order that the water may readily be reached by pumps, or perhaps in order that a flowing well may be secured. In the case of shallow wells the location is often affected by the possibility of local contamination, an element usually absent in the case of deep wells.

(C). Water collected in impounding reservoirs from streams of comparatively small watersheds depends for its good quality chiefly upon the scarcity of population upon the watershed. Suitable areas are therefore more likely to be found in the more rugged parts of the country and at the higher elevations, and usually at considerable distances, sometimes as great as 50 or 75 miles, from the population to be served. The location of such impounding reservoirs is also largely dependent upon questions of construction, such as the location of the dam, length and cost of aqueduct or conduit, and, what is of great economic importance, whether the water can be conveyed and distributed entirely or partly by gravity.

29. Works for the Distribution of Water. These include aqueducts and conduits for conveying water from a distant source, pumps and pumping stations, local reservoirs for equalizing the flow or for storage, and the pipes for distributing to the consumers. Conduits may be open channels, masonry conduits, or pressure conduits, such as pipes of wood, iron, or steel, and sometimes tunnels. The form is determined chiefly by considerations of cost. Pumps are used in a great variety of forms and situations, and may be operated by steam, gas, electricity, wind, or by hydraulic power. There are deep-well pumps for drawing water from depths not reached by suction, low-lift pumps for raising water from a river into settling basins or on to filters, or from wells into a low reservoir; and high-lift pumps for forcing the main supply into the distributing pipes or into an elevated distributing reservoir. Local reservoirs are used for receiving water from long conduits and regulating the flow in the distributing system, for equalizing the flow and pressure in pumping systems, and as settling reservoirs. The pipe system includes distributing mains, fire hydrants, service pipes, shut-off valves, regulating valves, etc.

30. Works for the Purification of Water. These vary in kind according to the nature of the impurities to be removed. Thus in the case of surface waters the sediment, bacteria, etc., are removed more or less completely by settling basins and various forms of filters. In the case of ground waters iron may be removed by aeration and filtration; hardness by chemical precipitation, etc. In these ways waters otherwise very undesirable can be greatly improved or made entirely satisfactory, but of course at a considerable expenditure of money. It will often happen, therefore, that a source of good quality but expensive will need to be compared with another poor in quality but capable of being made fairly comparable with the other at no greater total cost. Not infrequently the possibility of the future deterioration of a surface supply and the consequent necessity for artificial purification must also be considered.

RIVER AND LAKE INTAKES.

In drawing a water supply from a large river or lake a pipe or tunnel must extend from the pumping works out some distance from shore and the construction of such pipe line or tunnel often involves some very difficult work.

31. River Intakes. The location of the point of intake must be selected with reference to (1) the quality of water, and (2) the cost of construction and maintenance of the works connected therewith. The point of intake should be free from local sources of pollution and should therefore be located above all sewer outfalls of the town in question. In the case of tidal streams, sewage-polluted water may be carried long distances above the respective outfalls at flood tide, and before selecting the location careful study should be made of this question by means of floats and by examinations of the water at various seasons of the year. The location of the intake must also be determined with special reference to the lowest water stage.

The form of construction to be used depends upon the character of the stream in question, especially whether the difference between low and high water level is small or great.

If the water level vary only a small amount, as in the case of streams near dams or near a lake or ocean, the water may usually be taken from near the shore, the end of the intake pipe being supported on a small foundation of concrete, or on a wooden crib, or by a masonry retaining wall.

The intake pipes, usually of cast iron, may lead directly to the pumps, thus acting as suction pipes, or to a gate chamber and pump well. In the latter case the suction pipes of the pumps lead from this pump well. Gratings of cast iron or wood, with large openings, are usually placed at the entrance to the intake to prevent the admission of large objects, while fish screens of copper are inserted in the gate house or placed over the ends of the suction pipes.

If there is a large fluctuation of water level considerably more work is involved. It is usually necessary to extend the intake pipe a considerable distance from the banks of the stream in order to reach a suitable location at low water. Furthermore, pumps cannot lift by suction more than about 20 feet in practice, hence in order to enable the pumps to reach the water at the lowest stage, it is often necessary to place them in a deep pump pit much below high water level. The construction of a water-tight pit for this purpose is then an important feature of the works.

Another form of construction at the end of the intake is a masonry tower extending above high water and containing ports and sluice-gate similar in form to those used in reservoirs. To provide stability against ice and drift the tower is built similar to a bridge pier in form, the inlet ports being placed along the sides. The outer end of the intake pipe is usually protected by a simple timber crib supporting the end of the pipe 2 or 3 feet above the river bottom, and held in place and protected from scour by broken stone. A coarse screen or grating is ordinarily placed over that compartment of the crib containing the intake pipe. It is desirable to have the total area of the openings of this grating 2 or 3 times that of the pipe itself in order to keep the entrance velocity low. Sometimes in order to strain out the sediment the crib is entirely filled with broken stone and sand to form a filter crib. Such intake towers are used at St. Louis and at Cincinnati and tunnels connect with the tower through which the water is conveyed to the pumps.

The tower has the advantage over the crib construction in permanence and reliability. For these reasons this form of construction is to be commended, but it is much more expensive than the crib construction and is therefore suited only for the larger and more important works.

From the crib or inlet tower the intake pipe or tunnel usually runs to a screening chamber or pump well and from this chamber suction pipes lead direct to the pumps.

Fig. 5 illustrates a good design for small works. Here the water flows by gravity to the wet well made of boiler steel and constructed below high water line. From this well the water is drawn by suction pipes attached directly to pumps in the pump pit. The suction pipe is placed in a tunnel through which access may be had in time of high water to the valves in the wet well. The size of

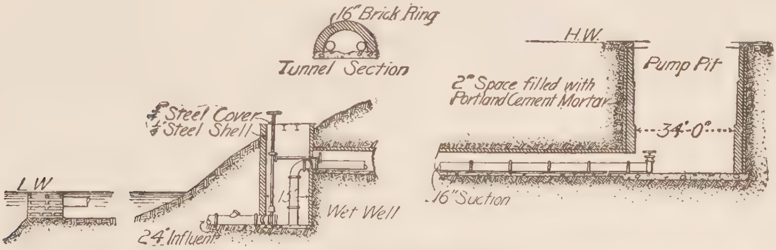


Fig. 5. Intake on the Ohio River.

intake pipe and suction pipe should be such that the velocity of the water in them will not exceed $1\frac{1}{2}$ to 2 feet per second.

32. Lake Intakes. The location of a lake intake in such a position as to obtain at all times water of the best quality, and to fulfill the requirements of safety against interruption, is a question requiring very careful study. In a lake unpolluted by sewage some of the things to be investigated are—the location of the mouths of streams and the sediment carried by them; the character of the lake bottom; the direction of wind and currents and their effects in stirring up the mud on the lake bottom and in conveying sediment from point to point.

The intake should if practicable be located at a sufficient depth to be free from any considerable wave action, both to secure a greater stability and to avoid the effect of the disturbance of the sediment by the waves. Even in small ponds the wind stirs up the water to a depth of 15 or 20 feet, so that this may be taken as about the minimum depth. A greater depth is desirable if the water is not too stagnant, since the water becomes rapidly cooler below this point. In large lakes the wave action extends to much greater depths and the intake should be extended accordingly to depths of 40 or 50 feet.

Most of the cities along the Great Lakes dispose of their sewage by running it directly into the lake at the most convenient point; and for those places that draw their water supply from the same body of water the most difficult part of the intake problem is to exclude their own sewage. As the cities grow, the intakes are pushed farther and farther out, but usually not until the necessity of the step is brought home by increased mortality from typhoid fever; and, however carefully this matter is followed up, the quality of the water taken from such sources must always be looked upon with suspicion. In Chicago the length of intake has gradually increased to 4 miles. In Milwaukee it is $1\frac{1}{2}$ miles, while the new intake at Cleveland is about 5 miles long.

Whether the conduit should be a pipe line or a tunnel depends upon the cost of construction and the relative reliability of the two forms. In small works the cost of a tunnel would be prohibitory, while in the case of a very large intake a tunnel may be the cheaper. Again, a pipe-line, unless sunk very deep, is subject to disturbances near the shore end by ice action, wreckage, and scour from storms.

Submerged-pipe intakes are usually laid by the aid of divers, although other methods have been used. The pipe is preferably laid in a dredged trench, at least as far out as wave-action is to be feared, and should be covered generally to a depth of 3 or 4 feet. Near the shore end the covering should be considerably deeper than this. Various methods of laying submerged pipe are described later.

Most lake intakes are protected at their ends by submerged crib work of timber partly filled with stone, the end of the pipe being raised 6 or 8 feet above the lake bottom to prevent the entrance of sand. At some of the larger ones, as at Chicago and Cleveland, large inlet cribs or towers built above the water surface are used, similar to river inlet towers.

The greatest difficulty met with in operating lake intakes is due to the clogging of the ports by anchor ice. This consists of needles and thin scales of ice which form in moving water and which are of such small size that they are readily carried below the surface by comparatively weak currents. They cease to form after the body of water has become frozen over. On coming in contact with submerged objects these particles of ice adhere and soon form large masses difficult to dislodge. Anchor ice has given much trouble at lake intakes both at the exposed cribs and at the shallower submerged ones. It is removed in various ways. Compressed air discharged near the port has been effective in some submerged intakes. Steam, water from hose, chains drawn back and forth through the ports, and pike poles, are some of the other means used. As tending to obviate difficulty with anchor ice all crib openings or port holes should have a large area so that the velocity of flow through them will not be more than 3 or 4 inches per second.

WORKS FOR THE COLLECTION OF GROUND WATER.

33. Collection of Water from Springs. The chief objects to be accomplished in the construction of works of the kind here considered are the protection of the water from pollution and the spring from injury through clogging or otherwise, the furnishing of a convenient chamber from which the conduit pipes may lead, and, in some cases, the enlargement of the yield by suitable forms of construction.

If a supply sufficient at all times for the demand can be obtained from one or more large springs, each one should have its separate basin from which the water may be conducted to a common main. The simplest form of works consists of a small masonry well or basin surrounding the spring and from which the conduit pipe leads. To prevent a growth of vegetable organisms and consequent deterioration of the water, such basin should always be covered so as to exclude the light. For a small spring, a circular well covered with a stone cap cemented in place and provided with a manhole is a simple and effective arrangement. For larger springs a masonry vault covered with 2 or 3 feet of earth is preferable. If the spring is located on a steep hillside, the collecting chamber is conveniently constructed in the form of a horizontal gallery built into the hill, access to which is had through a door or manhole.

Mineral and other springs occurring in public places usually have open basins, and opportunities are offered in the walls and parapets for ornamentation.

If the natural yield of a spring is insufficient, it will sometimes be possible to increase it. The proper form of collecting works to accomplish this depends upon the character of the spring. If the water appears at the upper surface of a stratum of impervious material overlaid by the water-bearing deposit, frequently in the form of several small springs, instead of dealing with each one individually it will often be better to construct a long collecting gallery running parallel to the outcrop and leading to a central collecting chamber which can be made similar in form to that for a large spring. This gallery, which may be made similar to that shown in Fig. 12, should be built deep enough to rest upon the impervious material, and thus to collect all the underground flowage as well as that appearing as springs. The total yield may be thus much increased, the increase being relatively greatest during dry weather.

The gallery may be simply a line of drain tile or vitrified pipe laid with open joints at the upper part of the pipe. If large quantities are collected the gallery may be made of brick or stone and large enough to permit of the passage of a man.

Where springs originate in a deep porous stratum such stratum may usually be tapped by wells without much reference to the spring. Use of such wells will generally reduce or entirely stop the flow of the spring.

THE CONSTRUCTION OF WELLS.

34. Principles Governing the Yield of Wells. If a well, either large or small, be sunk into a body of water-bearing material the water will run into such well, and if no pumping is done the water will, after a time, reach a level in the well the same as the level of the water in the surrounding soil. Fig. 6 represents a section through such a well. The dotted line A B represents the level of the water in the ground and in the well. Now if water is pumped from this well the level of the water therein will be lowered and as a consequence water will tend to flow into it from the surrounding ground and the surface of the ground water will assume some such shape as shown by the full line C D E F. The amount which the water surface is lowered decreases rapidly as we get farther from the well, until at some point more or less remote there is no sensible effect. The area within which the level is appreciably lowered is called the *circle of influence.* If the pumping is continued the level will be more and more lowered until it is so low that water will run into the well as rapidly as it is pumped out, after which no further change will take place. If the pumping ceases the well will gradually fill up to the original level.

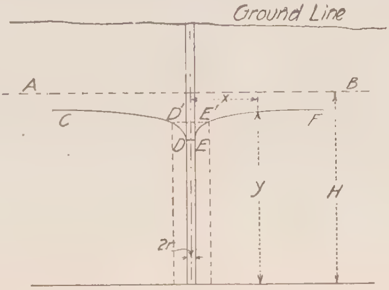


Fig. 6. Well Sunk in Ground Water.

Where the water flows under pressure, as in a porous stratum overlaid by an impervious one, the flow into a well is not accompanied by a change of *level* in the surface water, but the curve of *pressures* is of a form similar to the water surface in the case already treated.

The principles underlying the yield of wells have been investigated both theoretically and practically, but the subject is too difficult to be discussed in detail here. There are certain general principles, however, that are very important and which aid greatly to a clear understanding of the behavior of a set of wells under varying conditions. These may be stated as follows:

Having given a sand or gravel stratum of at least several feet in thickness in which water is flowing at some appreciable slope, such as 5 or 10 feet per mile, and a well is sunk into this stratum to a considerable depth, the yield of such a well when pumped from continually will follow approximately the following laws:

(1) The yield will be proportional to the distance the water level is lowered in the well below its normal level.

(2) The yield will be proportional to the thickness of the water-bearing stratum.

(3) For the same amount of lowering of the water the yield will be a little greater the larger the well, but the difference is not great except in case of very deep wells of small diameter in which the upward velocity of flow through the well is greater than 2 or 3 feet per second. A 10-foot well will yield only about 50 per cent more than a 6-inch well.

(4) For the same amount of lowering of the water the yield will be much greater in coarse material than in fine.

The following table will serve to give a rough idea of what may be expected from a single well sunk at least half way through a water-bearing stratum of various grades of material.

TABLE 9.

Approximate Yield of a 6-Inch Well When Sunk Into Water-Bearing Material 10 Feet Thick and When the Water Level Is Lowered One Foot by Continuous Pumping.

Material.	Yield in gallons per day.
Fine Sand	4,000
Medium Sand	30,000
Coarse Sand	80,000
Fine Gravel, free of Sand	500,000 or more.

For other thicknesses of material and other amounts of lowering the yield can be obtained by the law of proportion as stated above. The great increase in yield due to increasing coarseness of material is very marked and shows that for this very reason it is very difficult to make close predictions as to yield. Larger wells will give slightly better results.

Example. A well is sunk into a water-bearing stratum consisting of medium size sand to a depth of 30 feet below water level. What will be the yield if the water therein is pumped down 5 feet below its normal level?

By Table 9 the yield would be about 30,000 gallons per day for a 10-foot stratum and one foot of lowering. Hence for a 30-foot stratum and 5 feet of lowering the yield will be about 3 × 5 = 15 times as much, or 15 × 30,000 = 450,000 gallons per day.

If two or more wells penetrating to the same stratum are placed near together and simultaneously operated, the total yield will be relatively much less than the yield of a single well pumped to the same level. This mutual interference of wells depends in amount upon the size and spacing of the wells, upon the radius of the circle of influence of the wells when operated singly, and upon the depth to which the water is lowered by pumping.

The amount of this interference depends mainly upon the distance the wells are apart. It also depends upon the amount the water is lowered by pumping, and upon the general capacity of the stratum. If the water is lowered a considerable amount, such as 10 feet, the wells should be placed 200 to 400 feet apart in order that the interference be not too great. A small spacing like 25 to 50 feet will give an interference of a large amount,—often as great as 50 per cent in the case of 3 or more wells. That is to say, if 4 wells are placed 50 feet apart the total yield is not likely to be more than 50 per cent of the yield if these 4 wells were placed 300 or 400 feet apart.

Where it can be done, the best way to determine the capacity of wells is by actual tests conducted for a sufficient length of time to bring about a condition of equilibrium in the flow, but unless this condition is approximately fulfilled such tests are apt to be very deceptive. With a flat slope to the ground water a test may be carried on for weeks and even months, and the circle of influence will still continue to widen, resulting in a gradually decreasing yield. It may thus require years of operation to bring the conditions to a final state of equilibrium.

In the case of deep wells sunk into rock strata it is impossible to make an analysis of the conditions so as to be able to predict the yield. A pumping test is a necessity, but in this case also a very useful principle to remember is that of proportionality of flow to the lowering of the water level in the well. Thus if by pumping the level down 10 feet we get 200,000 gallons per day we may say with great certainty that the yield will be about 400,000 gallons if the water is pumped down 20 feet. In all cases this lowering of the water is to be measured from the level to which it rises when no water is pumped. In a flowing artesian well to get this level it is necessary to extend the casing above the ground as far as the water will rise, or to cap the well and determine this level by a pressure gauge.

35. Large Open Wells. As already explained, the yield of a well that is constantly pumped from is not much affected by its size. For other reasons, however, large wells are often advantageous.

The large well possesses a great advantage over the small well in its storage capacity. If the pumping is carried on at a variable rate, it thus acts to increase greatly the real capacity of the large well over that of a series of small tube wells. Furthermore, in the operation of the pumps there are many advantages in being able to get the entire supply from a single well, or from two or three large wells close together, chief among which is the avoidance of long suction pipes. The large well is also of great advantage where it becomes necessary to lower the pumps, as it permits the use of a more economical form of pumping machinery.

Trouble is often experienced in the small wells through clogging and the entrance of fine sand. This is largely avoided in the large well, as the entrance velocity of the water is very small. Opportunity is also given for the settling of fine material.

The chief disadvantage of the large well is in its great cost compared to the tube well for like yields. This disadvantage increases rapidly as the depth increases, and where it may be economy to construct a large well to a certain depth to serve as a pump pit it will usually be cheaper to develop the yield by sinking tube wells from the bottom, or by driving galleries therefrom, than by further sinking.

Large wells for waterworks are constructed of diameters of 10 feet or less to as great as 100 feet, 30 to 50 feet being the most common size. The minimum depth of a well is determined by the depth necessary to reach and penetrate for a short distance the water-bearing stratum, allowing a margin for dry seasons.

In the construction of a large well large quantities of water will be met with, and adequate means of handling it must be provided. As the water level must be kept at the lowest level of the excavation, the maximum pumpage will be considerably more than the future capacity of the well. For moderate depths the excavation can be carried on with no other aid than sheet piling. If the well is of large diameter, an annular trench is usually first excavated and the curb or lining built therein, after which the interior core is removed. This method enables the sheet piling to be readily braced. A method adapted to smaller wells is to drive the sheet piling outside of a series of wooden frames or ribs, and to excavate the entire well at once. The ribs are built in place as the excavation proceeds. This method is illustrated in Fig. 7.

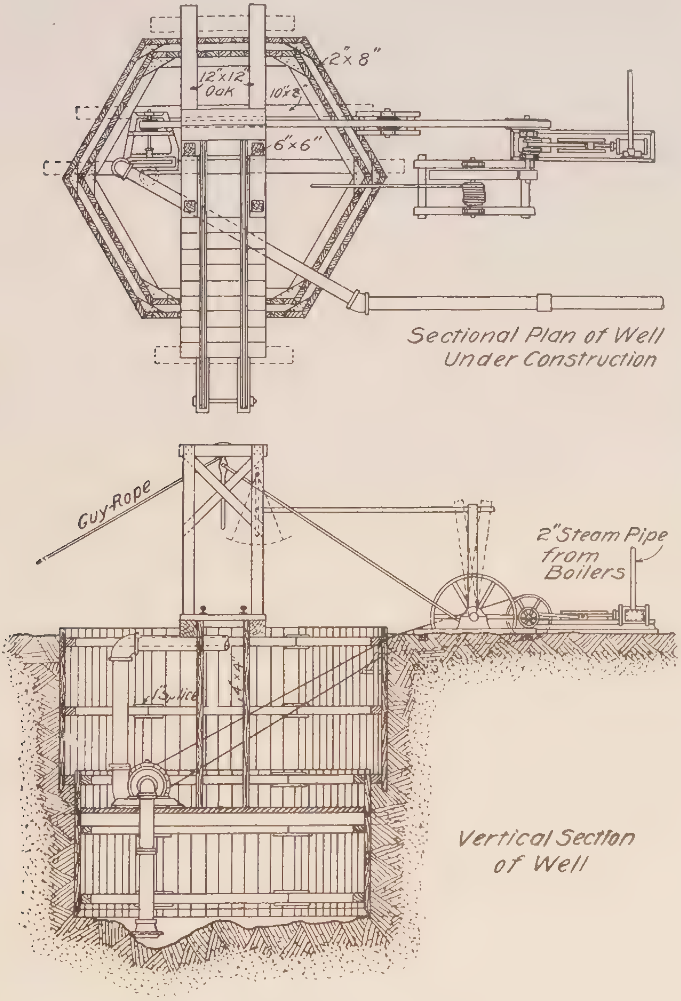


Fig. 7. Large Well under Construction.

For wells of considerable depth sunk in soft material, the curb may be started on a shoe of iron or wood, and the excavation and the construction of the curb carried on simultaneously, the curb sinking from its own weight. The material may be either excavated in the ordinary way, or by the use of compressed air, or dredged out without attempting to keep out the water, the method used depending upon depth of well, quantity of water, and character of the material. Where the friction becomes too great to sink the first curb the desired distance, a second curb with shoe may be sunk inside the former. In Fig. 8 are illustrated two forms of shoes used in sinking wells. These are both constructed mainly of wood. To strengthen such curbs iron rods should extend from the shoe well up into the masonry. For large wells, pump pits, etc., heavy iron shoes are often employed, and occasionally a pneumatic caisson is found necessary.

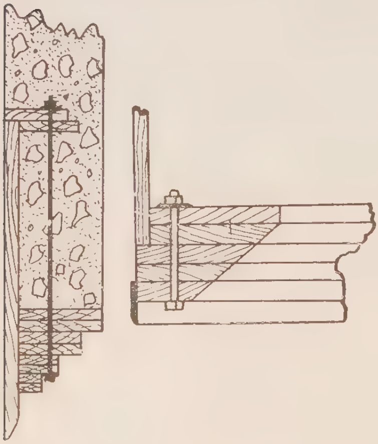


Fig. 8. Shoes for Sinking Well Curbs.

The lining or curb usually consists of a circular wall of brick or concrete masonry of a thickness varying with diameter and depth of the well, but ranging ordinarily from 2 to 5 feet. Dry rubble may be used for the lower portion, but the upper portion should be of concrete.

All wells should be covered to exclude the light and to prevent pollution of the water. The cover is usually made of wood, which for large wells may be conveniently made of a conical form and supported by a light wooden truss, or by rafters resting against the wall.

36. Shallow Tubular or Driven Wells. Shallow tubular wells, or wells of small diameter, also called *driven* wells, are sunk in various ways, depending upon the size and depth of well and nature of the material encountered. To furnish large quantities of water it usually requires a number of wells, and in addition to the question of sinking, questions of arrangement, spacing, connecting and operation are important. We will here consider only the methods of sinking wells in earth or soft strata.

As regards methods of sinking there are two principal kinds of wells—the closed-end well or driven well proper, and the open-end well.

The Closed-end or Driven Well. In this form the well tube consists of a wrought-iron tube from 1 to 4 inches in diameter, closed and pointed at one end, and perforated for some distance therefrom. The tube thus prepared is driven into the ground by a wooden maul or block until it penetrates the water-bearing stratum. The upper end is then connected to a pump and the well is complete. Where the material penetrated is sand the perforated portion is covered with wire gauze of a fineness depending upon the fineness of the sand. To prevent injuring the gauze and clogging the perforations, the pointed end is usually made larger than the tube, or the gauze may be covered by a perforated jacket.

Fig. 9 shows a common form of well point and the method of driving wells by means of a weight operated by two men. The tube may also be driven by a wooden block operated by a pile driver or other convenient means. Such a well is adapted for use in soft ground or sand up to a depth of about 75 feet, and in places where the water is thinly distributed.

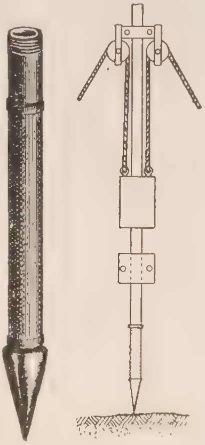


Fig. 9. Well Point and Driving Rig.

Open-end Wells. For use in hard ground and for the larger sizes the open-end tube is better adapted. This is sunk by removing the material from the interior, and at the same time driving the tube as in the other case. A very common method of sinking is by means of the water jet. In this process a strong stream of water is forced through a small pipe inserted in the well tube, the water escaping in one or more jets near the end of the pipe. At the same time the pipe, which is provided with a chisel edge, is churned up and down to loosen the material, which is then carried to the surface by the water in the annular space between the pipe and tube. If the material is hard or the well deep, a steel cutting edge may be screwed on to the end of the well tube.

With the open-end well the lower portion may be merely perforated with small holes in case the material is coarse or gravelly, or if sand is met with, the holes may be covered with brass gauze. Instead, however, of using a gauze it is common with this style of well to sink a solid tube, insert a special strainer of suitable length, and then withdraw the tube nearly to the top of the strainer.

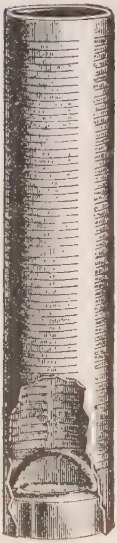
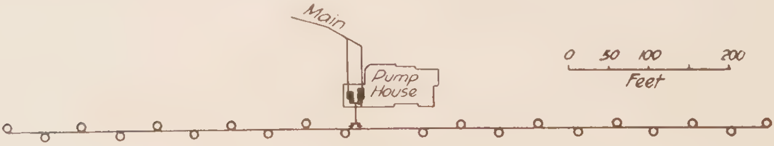


Fig. 10. Cook Well Strainer.

Fig. 10 illustrates a commonly used form of strainer known as the Cook strainer. It is made of brass tubing and provided with very narrow, slotted holes, which are much wider on the interior than on the exterior, an arrangement intended to prevent clogging.

Small tubular wells are usually arranged in one or two rows alongside a suction pipe and connected thereto by short branches. The smaller sizes are connected directly to the branch, the well tube acting also as a suction pipe, but with the larger sizes a separate suction pipe is ordinarily employed. In the former case, to avoid the entrance of air, it is necessary that the perforated portion of the pipe be always under water, and to insure this being the case it should be kept below the limit of suction. With the latter arrangement there are no such limitations to the position of the perforated well casing.

In order to enable the pumps to draw as much water as possible from the wells the pumps and suction main should be placed as deeply in the ground as practicable. A typical arrangement of wells is shown in Fig 11. In this plan the wells are 6-inch wells and are spaced 50 feet apart and are 35 to 50 feet deep.

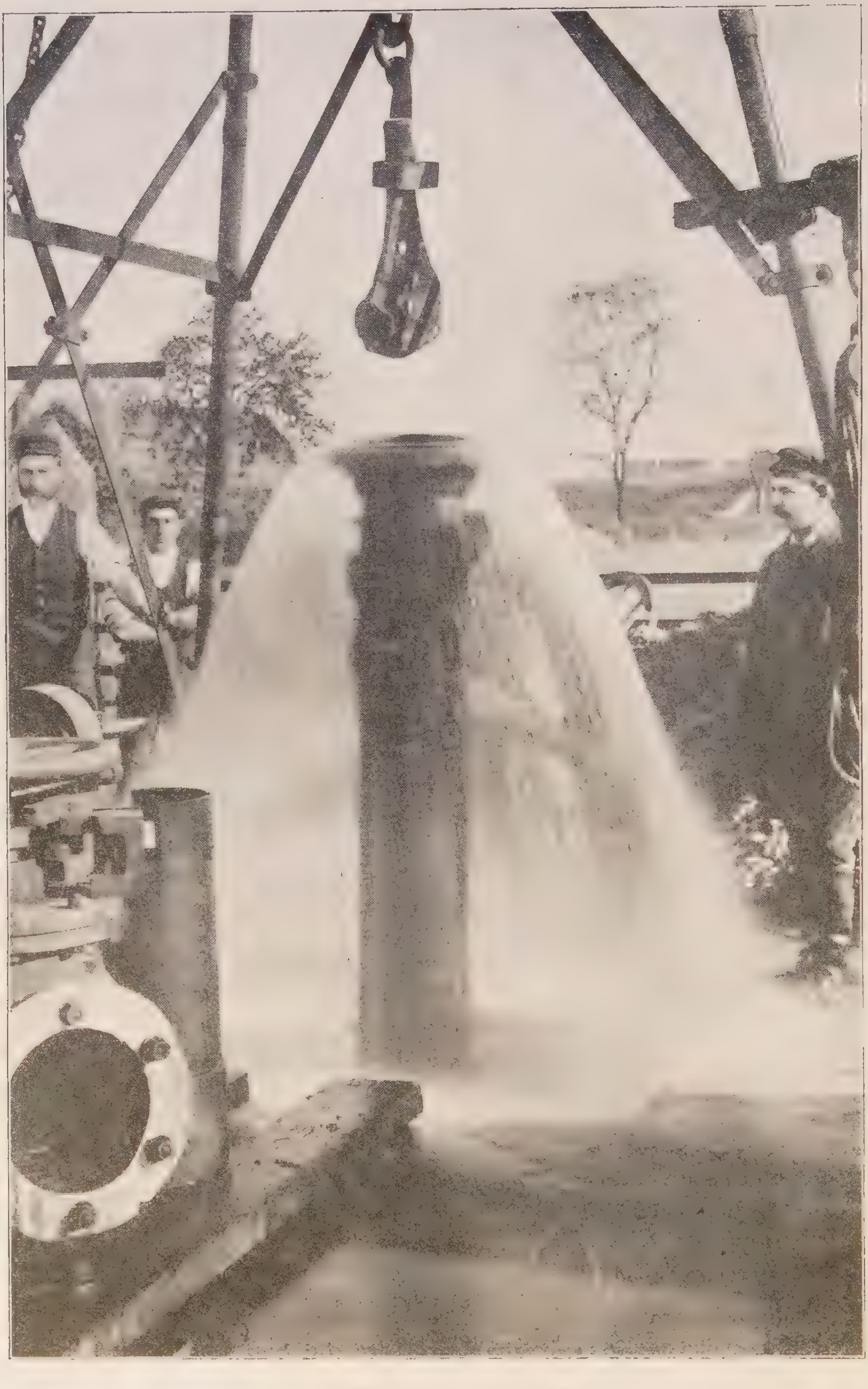


Plan

Fig. 11. Typical Arrangement of Wells.

SHOSHONE FALLS OF THE SNAKE RIVER, IDAHO

Height 210 feet. The bed of the Snake river canyon lies at a general level of 700 feet below the valley which its waters irrigate through the Twin Falls project. The Shoshone Falls, Twin Falls (187 feet), and Augur Falls (140 feet)—three of the six large falls in the canyon—all lie within five miles of the town of Twin Falls, center of an area of 250,000 acres of rich soil to be reclaimed. The waters are impounded by means of a dam at Milner, at the head of the canyon, 35 miles upstream from Twin Falls.



AN ARTESIAN WELL SPOUTING

View taken in Australia, showing a typical example of the spouting bores which are transforming vast areas of that continent from parched desert into the richest and most fertile of pastoral and agricultural regions. Australia has long been known as the driest of all the continents; but it has recently been discovered that the rock strata underlying the greater part of the country are storage reservoirs of inexhaustible water supply. Strangely enough, Victoria, which is itself watered by fine rivers, is devoid of these sources of underground supply.

The maximum amount of water obtainable from a given number of wells would be when they are spaced far enough apart so that their circles of influence will not overlap, but on account of cost of piping, and loss of head by friction, this would not be the most economical spacing. While it is impossible to give figures which would be of general application, it may be stated that from 25 to 100 feet is about the range for economical spacing of shallow wells. With very deep or artesian wells the spacing becomes still greater. Spacing less than 25 feet has quite often been used, but with doubtful economy.

Each well should be connected to the suction main by means of a short branch in which should be placed a gate valve, so that any well can be shut off at any time. The main suction pipe is usually made of flanged pipe, as this enables air-tight joints to be more readily made, although ordinary bell-and-spigot pipe with lead joints has been successfully used.

The greatest care must be taken in every part to make the work air tight, and to secure this it should be thoroughly tested in sections by means of compressed air. All valves should be carefully tested for air tightness, and all screw connections thoroughly fitted. In spite of the most careful construction, air will usually accumulate to some extent, and to eliminate it many plants are provided with air separators placed on the suction main near the pump. The simplest form consists of a large drum of steel placed on the suction pipe near the pumps through which the water passes at a slow velocity. A vacuum pump is attached to this drum.

Where sand is drawn up with the water it may be got rid of by passing the water at a slow velocity through a large drum or box inserted in the suction pipe and provided with suitable handholes for cleaning.

37. Deep and Artesian Wells. Where the depth exceeds 75 to 100 feet the small driven well is no longer practicable. Methods of sinking deep wells are in many respects different from those already described, and matters of spacing, pipe friction, arrangement of connections, etc., are much more important than in the shallow-well plant. Well boring is an art by itself, and the execution of any deep-well project should usually be put into the hands of some reliable well-drilling concern. The variety of ingenious tools and appliances in use for overcoming all kinds of difficulties and for penetrating all sorts of strata is very great, and it is possible to give here but a very general description of some of the methods of sinking in use.

In soft material it is necessary to case the well the entire depth, and on account of the difficulty of getting the casing down to great depths this operation becomes the chief feature of the construction. For depths up to 200 or 300 feet the ordinary well-drilling outfit can be used, and the casing driven close after the drill. When the casing can be driven no farther a smaller size is inserted and the sinking continued with a smaller drill, and so on until the well is sunk as far as desirable or possible. The material excavated is brought to the surface by means of a sand bucket, or by the water jet as previously described, the water being conducted to the end of the drill through hollow drill rods. By the latter method the hole is kept clean and a more rapid progress made.

The friction against the casing is greatly lessened, and the depth attainable much increased by the use of the revolving process. In this the lower end of the casing is provided with a toothed cutting shoe of hard steel of slightly greater diameter than the pipe, and the upper end is connected by means of a swivel to a water pipe through which water is forced by suitable pumps. The well is bored by turning the pipe, and the loosened material is carried to the surface by the water which passes down inside the casing and up on the outside between casing and soil. This process is very common in sinking artesian wells in the alluvial basins of California. It is very rapid, a rate of sinking as high as 20 or 30 feet per hour for depths of 1,000 feet having been attained.

It is essential to have a good length of strainer in the porous stratum. This is usually inserted after the desired depth has been reached, and the casing is then pulled up to the top of the strainer. By special devices it can, however, be attached to the end of the well casing and sunk with it.

A drilling outfit for deep wells is very similar to the ordinary familiar outfit for shallow wells worked by horse-power. A string of tools consists essentially of a steel bit, an auger-stem into which the bit is screwed, a pair of links or "jars" connecting the auger stem with another bar, called a sinker bar, and finally the rope cable which supports the apparatus and which passes over a pulley at the top of a derrick and then down to a winding drum. Just above the drum the cable is attached, by means of an adjusting or "temper" screw, to a large walking beam operated by a steam engine. As the work progresses the drill is lowerd by the temper screw. By means of the jars an upward blow may be struck to dislodge a jammed drill. Many ingenious tools are employed for recovering lost tools, cutting up and removing pipe, and carrying on the various operations involved.

Wells in soft material must be cased throughout. When bored in rock it is necessary to case the well at least through the soft upper strata to prevent caving. Casing is also desirable for the purpose of excluding surface water, to which end it should extend well into the solid stratum below. Where artesian conditions exist and the water will eventually stand higher in the well than the adjacent ground water, the casing must extend into and make a tight joint with the impervious stratum, otherwise water will escape into the ground above.

Ordinary artesian well casing is made of light-weight wrought-iron lap-welded pipe. For pipe which is to be driven the standard wrought-iron pipe is ordinarily used, but for heavy driving extra strong pipe is necessary. Joints of drive pipe should be made so that the ends of the tubing are in contact when screwed up. The life of a good heavy pipe is ordinarily very great, but cases have occurred where the pipe has been rapidly corroded, due to the presence of excessive amounts of carbonic acid.

The cost of sinking wells will of course vary greatly according to locality, nature of strata, and depth and size of well. For wells 6 to 8 inches in diameter and sunk in ordinary rock the cost per foot, not including casing, will usually range from $2.00 to $3.00 for depths of 500 feet, up to $3.00 to $5.00 for depths of 2,000 feet. For smaller sizes the cost will be somewhat less, especially for the shallow depths.

38. Connections for Deep Wells. The economical spacing for deep wells will be much greater than for shallow wells. It will likewise pay to spend more money in lowering the flow line by making deep connections, thus decreasing the number of wells and increasing the spacing. Generally speaking a spacing of from 400 to 800 feet will be found desirable.

On account of the relatively great cost of deep wells it will often be found economical to so arrange the pumps and connections that a considerable lowering of the water level below the ground surface may be obtained. This is generally accomplished by connecting all the wells to a single pump or set of pumps, placed at a considerable depth below the surface. Where the connections are very deep tunneling may have to be resorted to. Another common method of drawing water from deep wells in the case of small plants is by the use of a separate deep-well pump for each well. This method is applicable to any depth, but involves the use of uneconomical types of machinery. The air lift is another form suited to this work.

39. Yield of Artesian Wells. In making estimates regarding flow it is important to bear in mind that it requires a considerable length of time to determine with certainty the adequacy of the supply, and furthermore that the sinking of wells by other interests, even though at considerable distances, may very seriously affect the yield. Where conditions are sufficiently favorable for works of some magnitude the yield per well under a moderate head ranges from about 150,000 gallons per day to 800,000 gallons, or even more. With yields of less than 100,000 gallons per day, works for developing large quantities become very expensive, relatively more expensive than for small quantities, since with a large number of wells there is much greater interference. Often a well or set of wells will show a gradual falling off in capacity. The chief cause of a decrease in the yield of a well is the influence of other wells sunk in the vicinity. Where large numbers of wells are sunk in the same region this effect may be very serious, as in some cases where it has reduced the pressure of flowing wells from 75 or 100 feet down to nothing.

40. Galleries and Horizontal Wells. Where ground water can be reached at moderate depths it is sometimes intercepted by galleries constructed across the line of flow. If these are placed at a sufficient depth they will enable the entire flow of the ground water to be intercepted. In form a gallery may consist merely of an open ditch which leads the water away, or it may be a closed conduit of masonry, wood, iron, or vitrified clay pipe, provided with numerous small openings to allow the entrance of the water. Unless constantly submerged, wood should not be used. Masonry and vitrified pipe are preferable to iron, as these materials are uninjured by exposure to water. If galleries are not covered, excessive vegetable growth is apt to occur which may injure the quality of the water. Fig. 12 illustrates a form of gallery of concrete built in a water-bearing gravel.

Galleries for collecting ground water are occasionally tunneled in solid rock. This may happen along a side hill where an outcropping porous stratum overlies an impervious one and it is desired to develop the flow by running a tunnel along the hill near the bottom of the porous stratum; or it may occur where a steeply inclined artesian stratum can be more readily reached in this way than by vertical wells. Tunnels or galleries are also sometimes run from the bottom of large wells for the purpose of increasing the yield. This method of increasing the flow is advantageous where it is necessary to lower the pumps and to concentrate the flow in a single well.

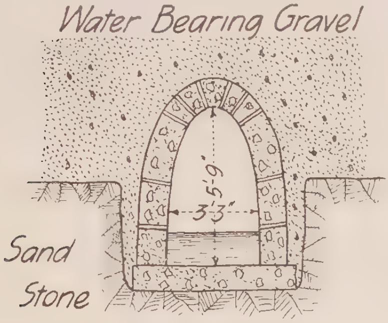


Fig. 12. Concrete Gallery.

Horizontal or push wells are tubular wells pushed approximately horizontally into a water-bearing stratum, or under the bed of a lake or stream. They are forced into the ground from a trench by means of jacks braced against the opposite side. These wells have been most successful when extended out under a lake or river.

Another method of utilizing a river bottom as a natural filter is to construct a wooden crib in an excavation in the bed of the stream, fill it with gravel and then cover the structure with 3 or 4 feet of sand up even with the river bottom. The suction pipe then leads from the crib to the pumps. This form of construction is well adapted to sandy-bottom streams with swift currents and has proved a very efficient way of clarifying muddy river waters.

Wells and galleries are often constructed near streams for the purpose of getting all or a portion of the supply therefrom. The success of such works depends much upon the character of the river bottom. Even when the lower strata are porous, the river, if a silt bearing one, may have a nearly impervious bottom and the natural filter will only become more clogged by use, necessitating perhaps the abandonment of the collecting works. Such failures have occurred in some instances. With a sandy river bottom kept clean by the scouring action of the floods, and with a porous substratum, works of this kind will give good results. To secure good filtration the works should be located at least 50 feet and preferably a greater distance from the stream.

The yield of a series of wells or of a gallery collecting filtered surface water will be, as in the case previously discussed, proportional to the lowering of the water level, and will be nearly proportional to the length of the line of works. In gallons per day per 100 feet of gallery, the yield from various successful works varies from 30,000 to 1,000,000 or more, which is about the same as is obtained from lines of wells.

RESERVOIRS AND DAMS.

Impounding Reservoirs.

41. Capacity. When the minimum flow of a stream is less than the daily demand of water it is necessary to store up the excess flow during the rainy season in large reservoirs called *impounding reservoirs.* The deficiency in the supply can then be made up by drawing from the reservoir. In this way the entire flow of a stream for a year or more may be stored and drawn off as wanted and streams that run dry at certain times may be made to supply quite a large population. Impounding reservoirs are made by constructing a dam across the valley in question, but natural lakes or ponds can often be utilized as reservoirs by building suitable works at this outlet.

In calculating the proper size of a reservoir we must consider (1) the yield of the source for successive intervals of time; and (2) the demand for all purposes for like intervals of time. The yield of the source of supply has been previously discussed. The demand to be considered includes not only the consumption for the city in question, but also the loss of water by evaporation from the area of the reservoir itself, also loss from leakage and percolation, and often the necessary withdrawals to satisfy the demands of riparian owners below.

The amount of leakage through the dam will usually be very small, but with certain forms of construction may be large. The quantity of water necessary to satisfy the demands of the riparian owners below the reservoir is often an exceedingly difficult matter to determine, and usually becomes a question for the courts to settle.

Practice differs greatly in different States, and in many of the Western States the water belongs to the State to dispose of as it sees fit. It is often expedient to buy up all rights and to utilize whenever necessary the entire flow of a stream, or to fix by contract the amount which will be allowed to flow.

The capacity of the reservoir must be based on the supplying capacity of the stream during the dryest year. The probable yield of the stream for each month of such a year should be estimated and recorded. Then likewise the monthly demand for the city in question and whatever allowance, if any, should be made for the use of riparian owners below.

Then for all months in which the demand is greater than the flow subtract the latter from the former; this will give the deficiency for each month. Add all deficiencies together and the result will be the total deficiency which must be made up from the reservoir and therefore is the required capacity of the reservoir, provided, however, that the total surplus for the remaining months is at least equal to the deficiency. If not, then the total yearly flow of the stream is not equal to the total demand and additional water must be obtained from some other source.

42. Location. In determining upon the location of a reservoir several elements must be kept in mind. In the first place it is very desirable that it shall be at such an elevation that at least a part of the consumers may be served by gravity alone, and it will be economy to spend a relatively large sum of money for conduits, or otherwise, to secure this advantage. The necessary elevation for this purpose depends upon the required pressure at, and elevation of, the various points of distribution, and the head lost in conducting thence the water.

The most favorable location for a reservoir as regards topography is a point where the valley forms a comparatively broad level area bounded by steep slopes at the sides, and below which the hills approach close together so as to form a good site for a dam. To prevent the escape of water the floor of the reservoir should contain no outcrop of porous strata of any extent which may lead the water away underground, and in the vicinity of the dam or embankment it should be underlain by an impervious stratum at a depth that can be reached by that structure.

After a tentative location has been decided upon, accurate levels must be run to connect the town with the reservoir site, also surveys for conduit lines, and an accurate topographical survey of the area to be flooded and all that may be affected by the reservoir. This survey should include information as to all buildings upon and adjacent to the area in question, nature of the vegetation, location of roads, property lines, etc. At the site proposed for the dam numerous borings must be made extending to a considerable distance above and below the dam as well as on the flanks, and these must be supplemented by test pits so that the nature of the supposed firm stratum can be accurately determined. If a suitable foundation cannot be reached at a reasonable cost, the site may have to be abandoned.

Calculations of storage volumes for different depths can readily be made from the contour map. The areas enclosed by each contour can be measured by a planimeter and the volume between any two successive contours taken as equal to the average of the areas enclosed by the contours, multiplied by the contour interval. The volume up to any given contour having been determined, the necessary height of dam to hold any given quantity of water becomes known.

All vegetation and perishable matter should be removed from the reservoir site, as the decay of such material injures the quality of the water. It is also desirable and of great benefit to the water to remove the top soil to a sufficient depth to include most of the organic matter therein.

As a further protection to the quality of the stored water it is desirable that there be as little area alternately flooded and exposed as possible, in order to limit the growth of vegetation. Shallow places should either be excavated to give a depth of 6 or 8 feet, or partly excavated and partly filled, the slopes being formed at about 3 to 1 and covered with sand or gravel.

43. Maintenance. In maintaining a reservoir so as to preserve the quality of the water and to supply the necessary quantity regularly and certainly requires a considerable degree of care and attention. To keep the quality as good as possible requires first of all that the watershed and reservoir be kept free from organic pollution. To insure that this is the case the city should have sani-

CANAL AND DAM AT SPIER FALLS, NEW YORK

At this plant, which is controlled by the Hudson River Power Company, about 30,000 horse-power is developed, which, combined with the output from other water power plants in the vicinity, is supplied to the shops of the General Electric Company at Schenectady, and also for light and power purposes to Albany, Troy, and other neighboring towns.

LA GRANGE IRRIGATION DAM, MODESTO, SOUTHERN CALIFORNIA

A typical example of work that is being done for the reclamation of arid areas in the West and Southwest. Under the direction of the U. S. Government Reclamation Service, irrigation projects are being carried out in thirteen States and three Territories. In every case the cost of the work is charged against the land, and thus ultimately paid back to the Government by the settlers. Among the most notable projects are the Salt river dam in Arizona, to irrigate 200,000 acres near Phœnix: the Yuma project on the lower Colorado river in California; the Klamath project, to reclaim 300,000 acres in California and Oregon; the Twin Falls project, on the Snake river in Idaho, to reclaim 250,000 acres; the Shoshone river project in Wyoming; the Gunnison river project in Colorado, to irrigate the Uncompahgre valley; and other projects in New Mexico, Oregon, Washington, Montana, and North Dakota.

tary supervision over the area in question, and inspection should be regularly made to see that all sanitary requirements are complied with. During seasons of low water, opportunity is offered for removing the vegetation from around the borders of the reservoir.

Careful records should be kept at the reservoir of all matters which may be of any value in subsequent designs for enlargement or for new works. These should include records of rainfall, temperature, height of water in reservoir, amount passing over waste weir, and data pertaining to the quality of the water at different seasons of the year. The maintenance of dams and embankments should call for very little labor. Earthen embankments should be kept neat in appearance with slopes well sodded, or covered with large gravel so as to be permanent. The top of the embankment should of course be maintained at its full height, and the waste weir and the channel below it kept clear and of the designed capacity at all times. Gates and other apparatus should be frequently inspected and kept in thorough repair.

EARTHEN DAMS OR EMBANKMENTS.

44. Kinds of Dams. Dams may be divided according to the material used into five classes; earthen dams, masonry dams, loose-rock dams, wooden dams, and iron or steel dams. These materials are also used in various combinations. The form of dam suitable for a given case depends upon the character of the foundation, the topography of the site, the size and importance of the structure, the degree of imperviousness required, and the cost. Of the above kinds of dams those of masonry and of earth are the ones usually considered.

The earthen embankment is the most common form of dam. It can be built on a variety of foundations; it is commonly the cheapest form, and when well designed and executed is an entirely safe and reliable structure. Where flood waters have to be passed over a dam some other material than earth must be used for at least the portion of the structure subjected to water action. Water flowing over an earthen embankment is inadmissible, many failures having been caused by such occurrence, due to faulty construction. For dams higher than 100 feet or thereabouts few engineers would recommend an earthen structure. If the foundations are suitable, a masonry dam is in such cases greatly to be preferred. It is more reliable, and with the great pressures occurring it is desirable to have all outlet arrangements built in masonry.

The general requirements of a good foundation for an earthen dam are that an impervious stratum can be reached at a moderate depth, and that the material near the surface is sufficiently compact to support the load. A compact clay or hardpan makes the best foundation. Solid rock is also good if not fissured. Embankments of earth have been successfully constructed on foundations of sand; but in such a case it is important that the sand be fine and of a uniform character, containing no streaks of coarse material which will offer little resistance to the flow of water.

Earthen dams are of a trapezoidal form with top width, side slopes, etc., proportioned according to the material used. Where good material is at hand in sufficient quantities the entire embankment may be made of uniform consistency and all as nearly water tight as possible. Usually, however, it will be more economical and give as good results to put the best material near the upper side of the embankment, changing gradually to the more porous materials towards the lower face. Where good material is scarce, imperviousness is usually obtained by means of a wall or "core" of impervious earth or masonry placed near the centre of the dam. If impervious foundation is reached only at a considerable depth, this portion only of the embankment is carried to the extreme depth.

Various kinds of material can be used to make an embankment. Loam, sand, gravel, and clay, mixed in various proportions, are common. For the first three to be impervious they must contain a certain proportion of clay, the amount required depending upon the variation in size of the coarser particles. The suitability of a material for embankment construction can to some extent be determined by experiments. It should be strongly cohesive and plastic when mixed with water, and should be impervious; but the correct valuation of natural mixtures requires much experience in their actual use in construction.

If good material does not exist already mixed, artificial mixtures of gravel, sand, and clay may be used. A fairly uniform sand or gravel contains about 40 per cent of porous space. If then a mixture be made of coarse gravel, fine gravel, and sand, in each case just enough of the finer material being used to fill the interstices of the next coarser, there will be in the mixture a porous space equal to .40 × .40 × .40 = 6.4 per cent, which will represent the proportion of clay necessary to make the mixture impervious. In practice it will take considerably more to insure the filling of all the interstices, as much as 15 or 20 per cent, depending upon the nature of the gravel mixture. In any case the percentage of voids in an artificial mixture can be readily determined by tests with water.

45. Core Walls. For a puddle wall of clay the minimum thickness ordinarily used is 4 to 8 feet at the top and about one-third the depth of water at the bottom, with a uniform batter on both faces. The trench is also usually made with a batter, the width at the bottom being one-third to one-half that at the ground level, with a minimum of 4 or 5 feet.

Instead of a core of puddle, many engineers prefer a core of rubble masonry or of concrete, made as impervious as possible. The advantages of this over a core of puddle are its safety against attack by burrowing animals, safety against wash in case minute leaks occur, and the greater certainty with which a concrete wall can be made impervious, especially where it joins the foundation.

Masonry core walls are made of various widths. Sometimes in case of embankments made of good material, they are made only a foot or two thick, their purpose being mainly to prevent the passage of burrowing animals. Ordinarily, however, a core wall is made 2 to 4 feet thick at the top, with a batter of $\frac{1}{2}$ to $\frac{3}{4}$ inch per foot on each side down to the trench and then with vertical faces below. The height of a core wall should be equal to that of the highest water level.

Figs. 13, 14 and 15 show cross sections of several forms of embankments. Fig. 13 is without core wall except in the trench, Fig. 14 has a core wall of puddle and Fig. 15 one of concrete.

46. Dimensions of Embankments. On the water side the slope is usually protected from wave action and should only be sufficient to prevent slips. With coarse material this need not be flatter than 2 horizontal to 1 vertical. With finer material it may need to be $2\frac{1}{2}$ or 3 to 1, or in some cases even 4 to 1, since earth in a saturated condition has a comparatively small angle of repose. On the lower side a slope of 2 to 1 is to be recommended, although $1\frac{1}{2}$

to 1 has frequently been used. If the material will stand at 1 to 1, as broken stone, for example, then a slope of 1½ to 1 would be suitable. On high embankments, bermes placed 30 to 40 feet apart vertically are a desirable feature.

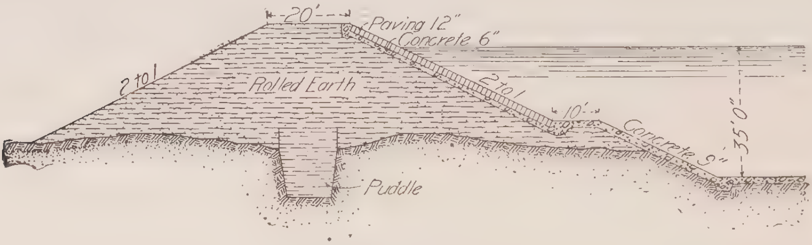


Fig. 13. Reservoir Embankment, Syracuse, N. Y.

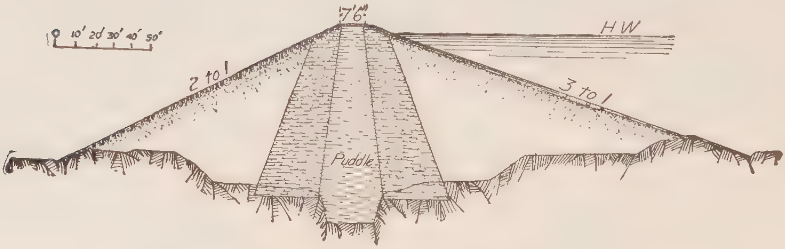


Fig. 14. Reservoir Embankment, Glasgow Waterworks.

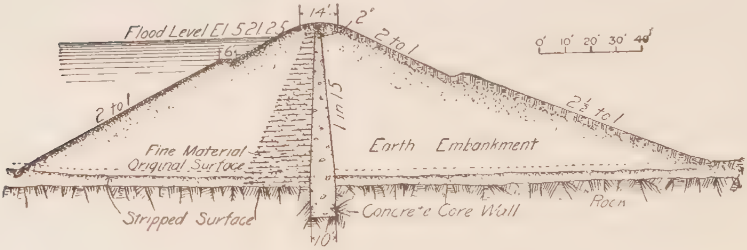


Fig. 15. Reservoir Embankment, Boston Waterworks.

The top of the dam should extend sufficiently above the high-water line to protect the material exposed to water action from frost and to give a safe margin against overflowing. This will be equal to the depth reached by frost plus an allowance of 2 to 5 feet for wave action, depending on the exposure to winds and the depth of the water.

The width of top is frequently fixed by the requirements for a roadway. Where not so fixed it is made to vary with the height—from 6 to 8 feet for very low embankments to 20 or 25 feet for embankments 80 to 100 feet high, or, approximately, width $= \frac{1}{5} h + 5$ feet, where $h =$ height of dam.

47. Construction. In preparing the foundation the surface soil must be removed over the entire site of the embankment to a depth sufficient to reach good sound material. All roots, stumps, and other perishable material must be removed, as any such material by decaying offers a passage for water. For the portion to be occupied by the core wall, if one is used, and a certain width in any case, the foundation must be excavated to an impervious stratum of solid rock or clay, and penetrate for a short distance such stratum. A sound bottom having been reached the surface should be roughened in order to give a better bond with the earth filling; and if the material is solid rock, all holes and crevices must be thoroughly cleaned and filled with cement or concrete.

After the foundation has been prepared the trench is first filled with the material selected. If puddle, it should be placed in 4- to 6-inch layers well rammed, or cut and cross cut with thin spades reaching well into the layer below, just enough water being used to render the material plastic. Where puddle is used in a narrow wall it is advisable to prepare it before placing by thoroughly pulverizing and tempering it with water, no more water being used than absolutely necessary. Puddle should be thoroughly worked and homogeneous. If concrete is used, special care must be taken to secure thoroughly good work in mixing and ramming, and in filling all irregular spaces in the excavation.

After the core is built to the surface, or a little above in the case of concrete, the main embankment is started. If the material used varies in quality, the finer and better should be placed above and adjoining the core wall, and the coarser placed on the down stream side and near the faces. If no core wall is used, the better material should still be placed in the up stream portion of the embankment. Stones exceeding 3 or 4 inches in diameter should not be allowed in the embankment except along the faces. The embankment is compacted usually by placing the material in layers 6 to 12 inches thick, wetting, and rolling with a heavy grooved roller weighing 200 to 300 pounds per lineal inch.

Much importance is attached to the work of compacting, and only by the best of supervision can work be secured. The use of water should be just sufficient to render the material plastic and capable of being packed, and no more. An excess of water makes rolling more difficult and increases subsequent settlement.

The up-stream slope must be protected from wave and ice action. This protection is usually afforded by a closely laid pavement about 18 inches thick laid on 6 to 12 inches of broken stone or gravel. Below low-water line a good layer of riprap is frequently substituted, the paving ending at a berme. The foot of the paving should be well supported by large blocks of stone or concrete. The down-stream face is usually sodded for sake of appearance and as a protection from rain, but may be protected by gravel and coarse material if more convenient.

48. Outlet Pipes. The design and construction of the outlet arrangements is one of the most important and at the same time most difficult features of the work. This is chiefly because of the difficulty of laying pipes or building masonry conduits through earth embankments in such a manner as to secure a perfect and reliable connection between the two materials. Poor work at this point is one of the chief causes of the many failures of earth embankments.

The outlet pipes are usually of cast iron and may either be laid underneath the embankment and surrounded thereby, or a culvert of masonry may be constructed in the embankment and the pipes laid therein, or they may be laid in a tunnel constructed in the natural ground at the end of the embankment or at some more remote point in the reservoir. A gate chamber containing the necessary valves is located at some point along the outlet pipe or conduit.

In the case of reservoirs with comparatively low embankments the outlet pipes are usually laid beneath the embankment at or near the lowest point. They should be laid on a good firm foundation in the natural ground, and should preferably rest upon and be surrounded by a bed of 8 to 12 inches of rich concrete, well rammed into the trench and left rough on the outside. To enable the earth to be more thoroughly bonded with the concrete, cut-off walls should be built projecting out from the main body of the concrete, 1½ to 2 feet, as shown in Fig. 16.

For some reasons an open culvert is much to be preferred to a simple pipe. Once constructed, additional pipes may be laid therein at any time; the pipes may also be readily inspected, and any leaks that occur in them do not endanger the structure, a matter of especial importance where the pipes are under heavy pressure. The same precautions must be taken in the construction of culverts as in the laying of pipes. They must have a good firm foundation and a good bond with the surrounding embankment. Imperviousness is secured by the use of a rich mortar and by plastering on the outside with Portland cement mortar neat or 1 to 1. Cut-off walls or projecting courses should be built around the outside at intervals as described for pipe outlets. At the connection with the gate house a cut-off wall is put in through which the pipes pass, and which must sustain the full head of water.

Figs. 16 and 17 show the two general methods of laying pipes through embankments.

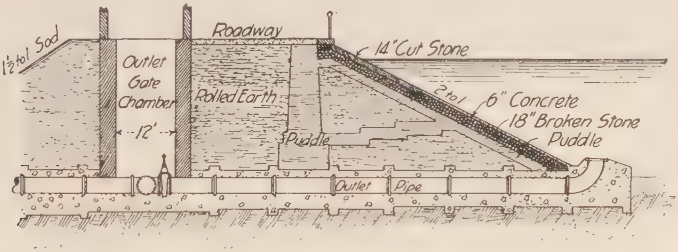


Fig. 16. Section Through Minneapolis Reservoir.

49. Gate Chambers. The gates or valves controlling the flow through the outlet pipes are placed in small masonry chambers, which, besides allowing of convenient operation of and access to the valves, also usually contain screening chambers and valve arrangements whereby water may be drawn from different levels. Gate chambers are preferably placed at or near the upper end of the outlet pipes in order that the pressure therein may be under control. They are, however, sometimes placed at the outer toe of the embankment, but this is undesirable, as it is impossible to shut off the water from the pipes in case of leakage except by the use of divers. Fig. 16 shows the gate chamber placed near the middle of the embankment, while Fig. 17 shows it placed at the upper end of the outlet pipe. One advantage of the latter arrangement is that water may be drawn from different levels so as always to get water of the best quality. Fig. 18 shows a gate chamber for a small works located as in Fig. 17.

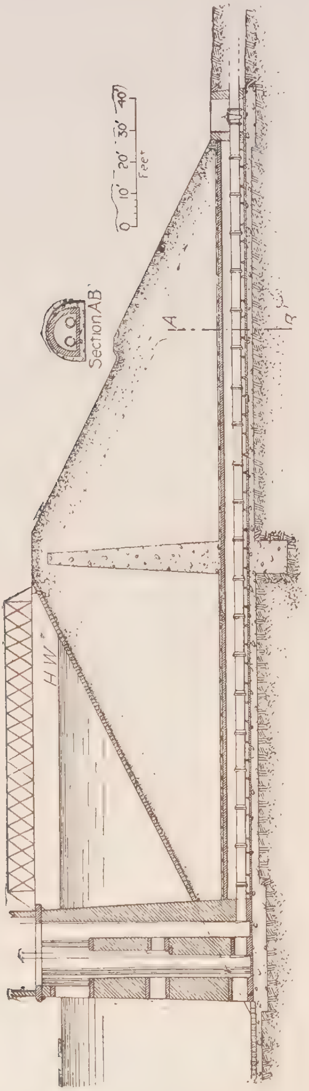


Fig. 17. Section Through Embankment and Gate Chamber.

The masonry of the chamber is usually of heavy rubble, faced with ashlar and lined with hard brick or cut stone. It should be laid in rich Portland-cement mortar to secure imperviousness. The walls will vary in thickness with their unsupported length, or the size of interior chamber, but the exterior walls are usually made 3 to 4 feet thick at the top, with an increase of about three-fourths inch to 1 inch in thickness per foot of depth, the batter being made on the outside for convenience and to furnish a better bond with the earthwork. Interior walls may be made of slightly less thickness. The foundation should be prepared with great care, as unequal settlement is liable to occur, causing cracks in the masonry of the culvert and displacing the outlet pipes.

Fish screens are usually copper-wire screens with $\frac{1}{8}$ to $\frac{1}{4}$-inch mesh, fastened to wooden or iron frames and arranged to slide in grooves in the masonry. They are arranged in pairs, and each screen is made up of several sections of a size convenient to handle.

The gate chamber is surmounted by a gate house in which is located the operating mechanism of valves and screens. As this building is frequently quite prominent, it is important that it be given an artistic treatment suited to the surroundings.

50. Waste Weirs. As already noted, one of the most fruitful causes of reservoir failures is insufficiency of waste weir capacity, resulting in the overflowing of the dam and its rapid destruction. Mention need only be made of the terrible Johnstown disaster in 1889, where, on account of insufficient wasteway, an earthen embankment was destroyed, resulting in the loss of over 2,000 lives and the destruction of property valued at 3 to 4 million dollars.

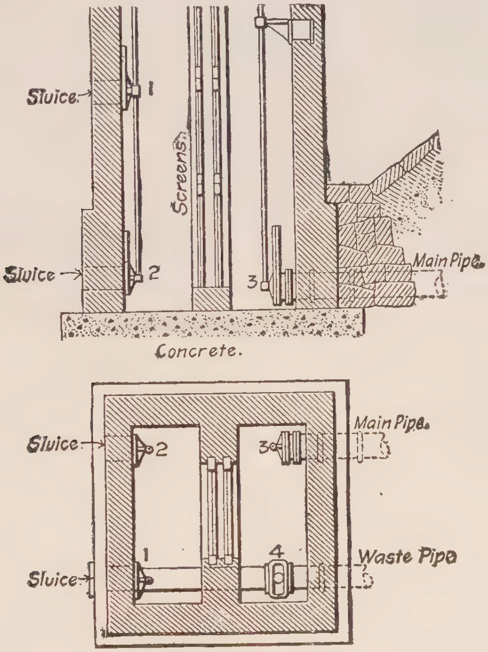


Fig. 18. Gate Chamber, Ipswich, Mass.

In section 14 the subject of maximum flood flows was fully discussed. The maximum flood having been estimated, it remains to provide some safe means whereby it may be passed to the valley below.

This is done in three different ways: (1) A wasteway may be excavated in the natural ground at one or both ends of the dam. Where the foundation is of rock this is a very safe and effective form of wasteway.

(2) The wasteway may sometimes be formed at some low point in the dividing ridge, and the water led to another valley.

(3) The third form of wasteway is provided by making a portion of the dam of masonry designed as a spillway, and placed at about the axis of the valley. The forms of such dams are discussed in detail in section 54. At the junction of the masonry and the earth portions, the lower slopes of the embankments must be retained by heavy wing walls built out from the masonry dam.

The requisite capacity being known, the length and depth of weir are to be determined. Either may be assumed and the other computed by means of the weir formulas as given in "Hydraulics." Weir heights will ordinarily range from 2 to 4 or 5 feet, with lengths of 50, 100, or even 500 feet, or more, depending on the required capacity. In any case the flood line determines the height of the main part of the dam, while the weir crest determines the storage capacity.

MASONRY DAMS.

51. General Conditions. Dams of masonry can safely be built only upon very firm foundations. Low dams of a height of 20 or 30 feet, and occasionally higher, have been founded on firm earth, but high masonry dams should be constructed on nothing less substantial than solid rock. In any case it is necessary to prevent practically all settlement, for with a material such as masonry any appreciable settlement is quite certain to cause cracks.

Masonry dams are designed in accordance with theoretical considerations so as to fulfill the following conditions: (1) The dam must not overturn or slide on its foundation, and (2) the pressures in the dam or foundation must be within safe limits.

The first consideration will govern the design of all dams up to a height of 100 feet or more and are therefore the only considerations which will be taken into account here.

Dams up to 30 or 40 feet in height are usually made trapezoidal in form, the saving obtained by making the faces curved or on broken lines not being enough to justify the extra trouble.

Let A B D C, Fig. 19, be a section of a trapezoidal dam. Let the dimensions be as represented in the figure. Further, let $w =$ weight of a unit volume of water, and $w' =$ weight of a unit volume of masonry. Let g = specific gravity of the masonry $= \frac{w'}{w}$.

The water pressure is represented by P and the weight of the dam by G, and it is assumed that the water level is at the top of the dam We will consider a length of dam of one foot. The height is known and the top width a and front batter m are assumed. Usually the front face A C is made vertical, or at a slight batter of one inch to the foot or so. In the former case $m = 0$ and in the latter $m = \frac{1}{12} h$.

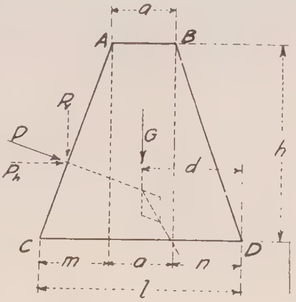


Fig. 19. Trapezoidal Dam.

From principles of mechanics we find the following value of the width of base l,

$$l = \sqrt{A + B^2} - B \qquad (3)$$

in which

$$A = a^2 + 2\,a\,m + \frac{h^2}{g} + \frac{m^2}{g}$$

and

$$B = \frac{m}{g} - \frac{m}{2} + \frac{a}{2}$$

In solving problems the numerical values of A and B should first be obtained and these values then substituted in the formula (3) for l.

If $m = 0$, as for a vertical face, then

$$l = \sqrt{\frac{5}{4}\,a^2 + \frac{h^2}{g}} - \frac{a}{2} \qquad (4)$$

Example. What width of base will be required if $h = 20$ ft.; $a = 5$ ft.; and the weight of masonry be considered 2.3 times the weight of water, or $g = 2.3$. Let value of m be 2 feet, giving a batter of 1 in 10.

Getting first the values of A and B we have $A = 5 \times 5 + 2$ $5 \times 2 + \frac{20 \times 20}{2.3} \times \frac{2 \times 2}{2.3} = 221$. $B = \frac{2}{2.3} - \frac{2}{2} + \frac{5}{2} = 2.37$.

Then $l = \sqrt{221 + 2.37 \times 2.37} - 2.37 = 12.7$ ft. Ans.

For dams exceeding 30 or 40 feet in height, it is economy to build the lower face in the form of a curve or broken line. The simplest way of calculating the section of such a dam (up to a height of 100 feet at least) is to treat it at first as similar to the form previously considered, but with a vertical upper face and top width of 0. Then the formula for bottom width becomes

$$l = \frac{h}{\sqrt{g}} \qquad (5)$$

This gives the triangular section A B C in Fig. 20. This form can then be modified by adding a suitable width *a* at the top and joining the point F with the sloping face A C by a smooth curve F D.

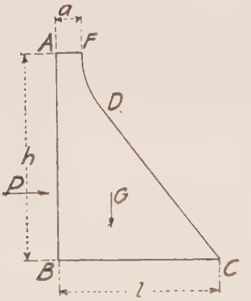


Fig. 20. Curved-Face Dam.

52. Top Width and Height Above Water Line. If the dam is to be used as a driveway, the top width will have to be at least 8 feet besides width of parapets. Otherwise the width and height above high-water line must be such as to secure stability against wave and ice action as just noted, and to prevent waves from washing over the top. In practice the width varies from a minimum of 4 to 5 feet for low dams to 15 or 20 feet for very high dams; and the height above high-water line from 2 or 3 feet to about 10 feet. In some cases much larger dimensions may be required for low dams than those given.

53. Construction. For large dams the foundation should be solid rock. In preparing the foundation surface all loose and partially decomposed material should be excavated until a firm base is reached. If the bottom is smooth it should be roughened by excavating shallow cavities in the rock. At points where crevices occur the excavation must be carried down to a solid bottom and all loose material must be removed. After an acceptable surface is reached it should be thoroughly washed or scrubbed with water in order that there may be a secure bond between the foundation and the masonry.

Uncoursed rubble or concrete is usually employed in dam construction. The object to be attained is to secure a homogeneous structure, free from all through joints or weak places of separation.

Concrete, well placed, is in this respect an ideal material. Rubble masonry, in which all joints are thoroughly filled with mortar, and larger spaces with concrete, has been used for most of the high dams.

In constructing the masonry the principal points to be emphasized are clean surfaces, irregular surfaces, joints absolutely filled with compact mortar, great care to give good bedding, and constant supervision. Mortar and cement should be thoroughly rammed into all spaces, using for this purpose suitable forms of rammers.

In the construction of dams of moderate height, earth backing is often carried up to the water level with a slope of 2 or 3 to 1, as in an earthen dam. If a dam is located on a porous or bad foundation or on one of earth, a good, compact backing will much reduce the percolation under the dam, and therefore the tendency of any upward pressure, and will add considerably to the safety of the structure. It is especially applicable to spillways in earthen embankments.

The arrangements for drawing water from the reservoir are similar in general to those described in the last chapter. The outlet pipes are built in the masonry at or near the lowest point of the dam, and terminate in a gate chamber constructed just above and in connection with the dam. The gate chamber has the same functions as explained in the case of earthen embankments. No danger is here to be apprehended from constructing the pipes in the body of the dam.

54. Masonry Waste Weirs. Masonry dams are not usually designed to allow water to pass over their entire length, but a certain portion only is made to act as a waste weir. The form of a masonry weir depends much upon local conditions, chief of which are height of dam, character of foundation, amount of ice and driftwood to be expected, and quantity of water to be provided for. A weir is essentially a dam with its top and lower face so constructed as to permit the water to pass over it without damage. Besides the design of the profile, the protection of the stream bed below the dam is a very important feature, as many dams have been undermined by failure at this point even where the bed has been solid rock.

Figs. 21, 22 and 23 show three forms of waste weirs. Fig. 21 permits the water to fall vertically and is suitable for small heights; Fig. 22 is preferable for larger quantities of water and greater heights; while Fig. 23 represents a form of construction suitable for the largest dams.

55. Timber Dams. Where a dam is constantly submerged, a timber structure is of a permanent nature, and will need repairs only on account of the wear of the apron. A timber dam may also be advisable in certain circumstances even when its life will be short, as, for example, where a temporary supply may be furnished pending the construction of more permanent works, or where the expense of permanent and costly structures is for the present prohibitory. Such dams are, however, used mostly for diversion purposes or for water power, and seldom for the storage of large volumes of water.

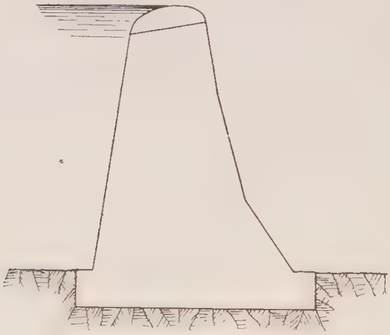


Fig. 21. Waste Weir.

Timber dams may be constructed on any kind of a foundation, but are usually built on rock or on a gravelly bed. They consist of cribs or frames built of logs or squared timber, filled with stone and clay, and planked over to render them water-tight. They may be built as separate cribs in sections, each section consisting of perhaps 3 to 4 cribs, or as one continuous framework. The former method is especially useful in dealing with large flows and irregular foundations, the stream being gradually closed as the sections are constructed. The cribs may also be filled and sunk separately so as to form piers on which a continuous structure may be built.

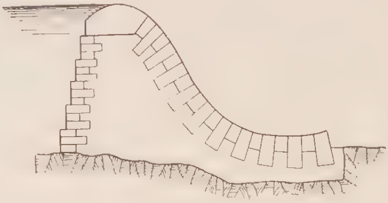


Fig. 22. Waste Weir.

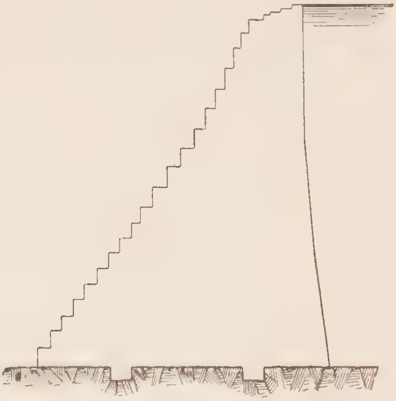


Fig. 23. Waste Weir.

The foundation of a crib dam, if soft, is prepared by dumping stone over the area to be built upon. In the framed dam the foundation must be more carefully prepared. Where it is soft the dam is supported on piling, and sheet-piling is used to prevent underflow. If the dam is built on a rock bottom, it must be bolted thereto. The framework is usually built with a sloping upper face and a series of stepped aprons below, or a single free wall to a water cushion. Rock and gravel, or puddle, is used for filling. Fig. 24 shows a form of dam on pile foundations and Fig. 25 a dam anchored to solid rock.

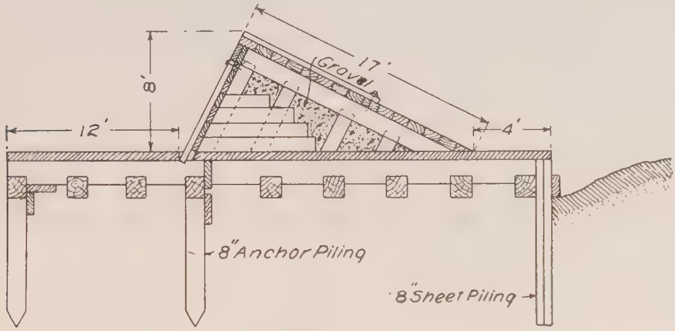
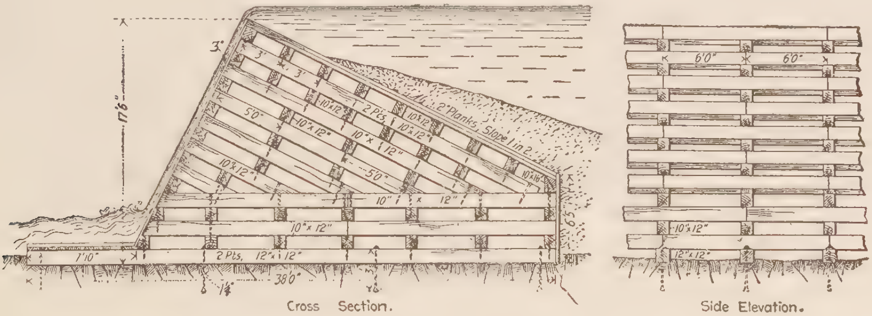


Fig. 24. Timber Weir.

56. Loose Rock Dams. Dams composed largely of loose rock have been used to a considerable extent, and in some respects present considerable advantages as to stability. Another advantage is that they can be constructed in running water, but the finished dam is not suited to act as a waste weir.



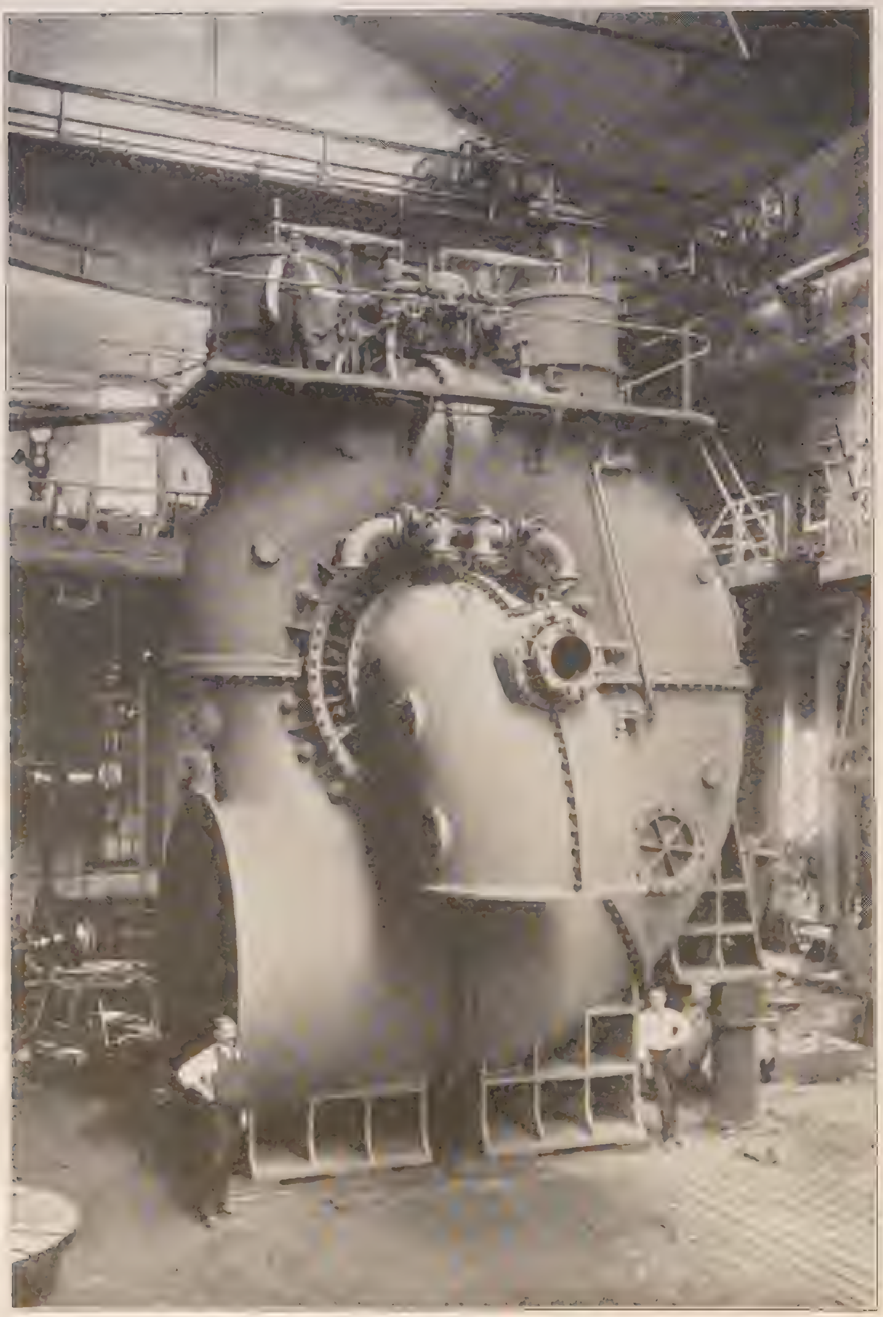
Cross Section. Side Elevation.

Fig. 25. Dam Anchored to Rock.

The body of the dam is made of loose rock placed with more or less care, and rendered comparatively impervious by a sheathing of plank, or by a facing of earth or fine material on the upper face, or, as in one case, by a core of steel. As regards stability the principle of construction is of the best. Since considerable percolation is likely to take place, such a dam cannot be founded on a material liable to scour; and if the dam is high, the foundation should be solid rock. The lower slope is usually 1 to 1, while the upper slope may be made $\frac{1}{3}$ or $\frac{1}{2}$ to 1; but to secure these steep slopes it is necessary to lay the stone for a considerable thickness as a dry wall. Above this wall the facing of timber or earth is placed. The former material is objectionable on account of its perishable nature.

UNRIVALED NIAGARA

Source of supply for power purposes now extensively used on both Canadian and American sides of the river. View from Prospect Point looking across brink of American Fall toward Goat Island; Horseshoe Fall in the distance.



LARGEST TURBINE IN THE WORLD

Installed at the Shawinigan Falls Power Station on the St. Maurice river, about 84 miles northeast of Montreal, Que. Weight, 364,000 pounds; height, 30 feet; weight of wheel, 5 tons; weight of 22-in. shaft, 10 tons; intake, 10 feet in diameter; amount of water passing through per minute, 395,000 gallons.

WATER SUPPLY.

PART II.

CONDUITS AND PIPE LINES.

PIPES.

57. Materials and Stresses. Where the source of supply is at a considerable distance from the place of consumption the design and construction of the necessary works for conducting the water is a matter of great importance and demands special consideration. Usually a distant source is at a higher elevation than the city to be served, so that it will be possible to convey the water partly or wholly by gravity. In many cases, however, a part or the whole of the water will require pumping, so that the design will also involve a study of possible pumping arrangements. It will usually be necessary to consider several designs based upon different locations and often upon different types of conduits.

A variety of materials may be employed for the construction of water conduits. If the conduit is not under pressure, the form of construction used may be an open canal dug in the natural earth, or a masonry conduit in "cut and cover," or a tunnel. Where the water flows under pressure the first two types are not suitable and a pipe, or possibly a tunnel, must be employed.

The materials used for water pipes are cast iron, wrought iron, steel, wood, cement, vitrified clay, lead, and occasionally a few other materials. The important requirements for a water pipe are strength, durability, and low cost. The relative importance of these requirements will vary under different circumstances, and this will lead to the use of different materials in different cases.

The tensile stress in a water pipe under pressure is given by the formula in section 10, of Hydraulics,

$$S = \frac{pr}{t} \qquad (6)$$

where S = tensile stress per sq. inch
p = pressure in lb. per sq. in.
r = radius of the pipe in inches,
t = thickness of pipe in inches.

If f represents the safe tensile strength of a pipe material, then the required thickness to resist the pressure will be given by the formula

$$t = \frac{pd}{f} \qquad (7)$$

Example. What will be the required thickness of a steel pipe 3 ft. in diam. for a water pressure of 100 lb. per sq. in. assuming the safe stress = 10,000 lb. per sq. in.

From formula 7, $t = \frac{100 \times 18}{10,000} = .18$ inch. Ans.

Besides the stress due to steady water pressure, the pipe must be strong enough to resist the shocks due to the sudden stoppage of flowing water, called *water hammer*, also the pressure of the surrounding earth and the action of other outside forces, changes of temperature, and blows and shocks received in transportation and construction. The stresses due to these additional forces cannot be accurately calculated; but they are allowed for in practice by various empirical rules.

58. Cast-Iron Pipe. Cast iron is the most widely used material for water pipe. By reason of its moderate cost, its durability, and the convenience with which it may be cast in any desired form it is almost universally employed for the pipes and various special forms of distributing systems. It is also frequently employed for large pipe lines, and is now easily obtained in any desired diameter up to 6 feet or more. Cast-iron pipes are made in lengths of about 12 feet, which are joined together usually by the bell-and-spigot joint run with lead. Branches and other irregular forms are used for connections. These are called special castings, or simply "specials."

A formula for the thickness of cast-iron pipe applicable to diameters up to 3 feet is as follows:

$$t = \frac{(p + 140 - 4\,r)}{3300} r + 0.25 \qquad (8)$$

where t = thickness in inches;
p = static pressure in pounds per square inch;
r = radius of pipe in inches;
0.25 = allowance for eccentricity, deterioration, and safety in handling.

In Table No. 10 are given the thicknesses of pipe for various pressures, also the average weight per foot, and the total weight of 12-foot lengths, as employed by the Metropolitan Waterworks, of Boston.

TABLE 10.

Thickness and Weight of Water Pipe.

Diameter in Inches.	Class A. 115-foot Head.			Class B. 150-foot Head.			Class C. 200-ft. Head.			Class D. 250-ft. Head.			Class E. 300-ft. Head.		
	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.	Thickness of Shell in Inches.	Average Weight per Foot.	Weight of 12-ft. Length.
4	....	......	......	....	......	....	....	....	....	0.40	19.2	230	0.45	21.2	255
6	....	......	......	....	......	....	....	....	....	0.46	31.7	380	0.50	34.6	415
8	....	......	......	....	......	....	....	....	....	0.52	47.1	565	0.55	50.0	600
10	....	......	......	....	......	....	....	....	....	0.60	66.7	800	0.63	70.0	840
12	....	......	......	0.57	75.8	910	0.61	80.8	970	0.65	85.8	1030	0.69	91.2	1095
14	....	......	......	0.61	94.2	1130	0.65	100.0	1200	0.70	107.5	1290	0.75	115.0	1380
16	....	......	......	0.65	115.0	1380	0.70	123.7	1485	0.75	132.5	1590	0.81	142.9	1715
20	....	......	......	0.73	160.8	1930	0.79	173.3	2080	0.85	186.2	2235	0.92	201.2	2415
24	....	......	......	0.80	210 4	2525	0.87	228.3	2740	0.95	248.7	2985	1.03	269.2	3230
30	....	......	......	0.92	302.1	3625	1.00	327.5	3930	1.10	361.2	4335	1.20	393.3	4270
36	0.93	366.7	4400	1.03	404.2	4850	1.13	441.7	5300	1.25	491.7	5900	1.36	533.3	6400
42	1.01	467.5	5610	1.14	524.2	6290	1.27	581.2	6975	1.40	645.8	7750	....	......	......
48	1.15	605.8	7270	1.25	655.8	7870	1.40	730.0	8760	1.55	818.3	9820	....	......	......
54	1.23	730.8	8770	1.35	797.5	9570	1.53	919.2	11030	....	......	......	....	......	......
60	1.35	885.8	10630	1.50	979.2	11750	1.70	1132.5	13590	....	......	......	....	......	......

59. Joints. The joint which is ordinarily employed in this country is the bell-and-spigot joint. The space between bell and spigot is filled with lead, which is calked solidly into place so as to be water-tight. Many forms of bell or socket have been devised, but practice has come to be quite uniform on this point.

In Table No. 11 are given various dimensions of standard bell and spigot of the Metropolitan Waterworks (Fig. 26), together with amounts of lead and packing required per joint.

The ordinary bell-and-spigot joint with lead packing will enable pipes to expand and contract under moderate changes of temperature such as occur with buried pipes.

Curves of large radius can be constructed with straight pipe by deflecting each length slightly. In this way it is possible, with a reasonable deflection, to lay 4 to 8-inch pipe to a curve of 150-foot radius, a 16-inch pipe to a 250-foot radius, and a 36-inch pipe to a 500-foot radius.

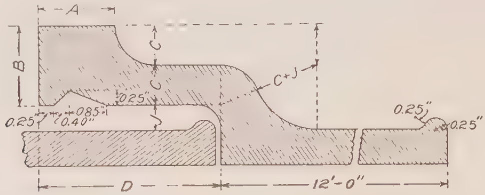


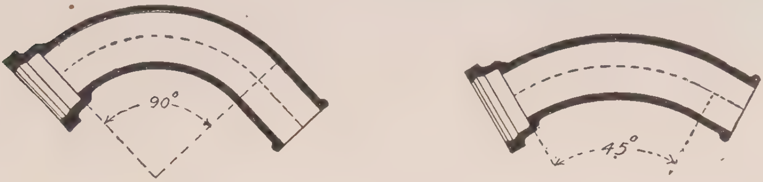
Fig. 26. Standard Bell and Spigot.

TABLE 11.

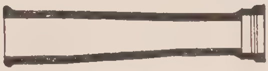
Dimensions of Standard Bell and Spigot, Metropolitan Waterworks, Boston (Fig. 26).

Size of pipe.	Class.	Dimensions in inches.					Average wt. of lead per joint.		Weight of jute gasket per joint.
		A	B	C	D	J	With gasket.	Solid lead.	
4	All classes	1.50	1.30	0.65	3.00	0.40	7	9¼	.10
6	"	1.50	1.40	0.70	3.00	0.40	9¾	12¾	.15
8	"	1.50	1.50	0.75	3.50	0.40	12½	18¾	.25
10	"	1.50	1.50	0.75	3.50	0.40	15¼	23¼	.30
12	"	1.50	1.60	0.80	3.50	0.40	18	27	.35
14	"	1.50	1.70	0.85	3.50	0.40	20½	31	.40
16	"	1.75	1.80	0.90	4.00	0.50	31¼	50½	.65
20	"	1.75	2.00	1.00	4.00	0.50	38½	62	.80
24	"	2.00	2.10	1.05	4.00	0.50	45½	74	.95
30	B and C	2.00	2.30	1.15	4.50	0.50	56	100½	1.55
30	D and E	2.00	2.50	1.25	4.50	0.50	57	102	1.55
36	A, B, and C	2.00	2.50	1.25	4.50	0.50	67	120½	1.85
36	D and E	2.00	2.80	1.40	4.50	0.50	68½	122½	1.85
42	A, B, and C	2.00	2.80	1.40	5.00	0.50	77½	154	2.60
42	D	2.00	3.20	1.60	5.00	0.50	78½	156	2.60
48	A, B, and C	2.00	3.00	1.50	5.00	0.50	88½	176	3.00
48	D	2.25	3.50	1.75	5.00	0.50	89½	178	3.00
54	A and B	2.25	3.10	1.55	5.50	0.50	99½	215	3.95
54	C	2.25	3.90	1.95	5.50	0.50	100	215½	3.95
60	A and B	2.25	3.20	1.60	5.50	0.50	110½	239	4.40
60	C	2.25	4.20	2.10	5.50	0.50	111	241	4.40

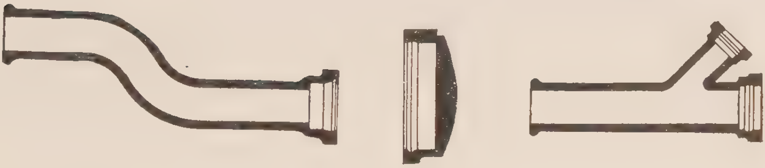
60. Special Castings. The ordinary special castings required are the $\frac{1}{4}$, $\frac{1}{8}$, and $\frac{1}{16}$ bends or curves, T's and crosses, or three-way and four-way branches, Y branches, blow-off branches, offsets, sleeves, caps, and plugs. The various forms are illustrated in Fig. 27. Many of the larger cities have their own standard designs for specials as well as for straight pipe, which differ more or less from the manufacturers' standards. For the smaller cities it will be much the more economical to use either the manufacturers' standards or those of some neighboring large city.



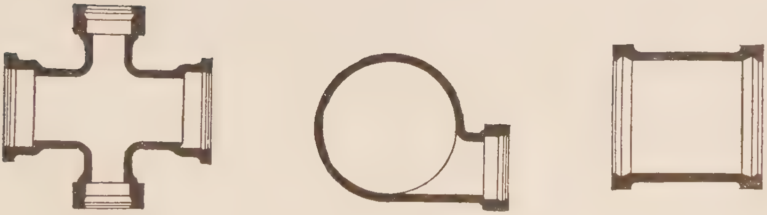
$\frac{1}{4}$ Bend. $\frac{1}{8}$ Bend.



Reducer.



Offset. Cap. Y Branch.



Four-way Branch. Blow-off Branch. Sleeve.

Fig. 27. Special Cast-iron Pipe Fittings.

The various branches are manufactured either with part bell and part spigot ends, or with all bell ends. The latter form is usually preferred for branches, as it enables connections to be readily made by means of pieces of pipe.

61. Wrought Iron and Steel Pipe. Wrought iron and steel have been used to a considerable extent for water pipes, and for large pipe lines these materials present considerable advantage over cast iron. Since steel is much stronger than cast iron, the use of it will give a much lighter pipe, an advantage as regards transportation, but a disadvantage as regards durability, especially for small sizes. Special forms are not so readily constructed of steel, so that for distributing mains cast iron is much preferable. Another disadvantage of steel pipe is that with the ordinary riveted joints a considerably larger pipe is required than if a smooth cast-iron pipe is used on account of the increased friction.

Small sizes of pipe may be made by means of the lap-welded joint, or the spirally-riveted joint, or the longitudinal lap-riveted joint. Such pipes are made in sections of 12 or 15 feet which are connected in the field in various ways, such as by a screw coupling, or by means of a cast-iron bell and a spigot consisting of a steel or wrought-iron band, or by riveting, or by merely driving the sections together. For large sizes riveted longitudinal and circular joints are usually employed. Single sheets are bent and riveted to form one section of pipe, which may be made either cylindrical in form, or made with a slight taper and the sections put together stove-pipe fashion. The design of the riveting is too large a subject to be taken up here.

Changes in direction are usually made by forming one or more joints at a small bevel. Two or three standard bevels of small angle may be adopted, and any desired curve made by the use of one or more of these bevels. Branches, etc., for the ordinary sizes of pipes, are usually made of cast iron and are riveted or bolted firmly to the steel pipe. Valves are joined to the pipe in a similar manner by means of cast-iron flanges.

62. Wooden Pipe. The manufacture of bored pipe for water mains has been somewhat revived in recent years, and a considerable amount of such pipe is now manufactured under the name of "improved Wyckoff pipe." The pipe is made from solid logs, but it depends for strength upon spiral bands of flat iron which are wound tightly about it from end to end. The exterior of the pipe is coated with pitch as a protection to the bands. The joints are made by means of wooden thimbles fitting tightly in mortises in the ends of the pipe, and, in laying, the sections are driven together by means of a wooden ram. The interior surface is smooth and continuous.

The pipe is made in sections 8 feet long, and in sizes from 2 to 17 inches in diameter. The bands are spaced according to the pressure. Branch connections are made by means of cast-iron specials which have long sockets into which the wooden pipe is driven. About 1,500 miles of this pipe is reported to be now in use. It is very durable and is said to cost somewhat less than cast iron where the transportation charges are not excessive. Wooden stave pipe is another form that has been extensively used in the West.

The durability of wooden pipe is chiefly a question of the life of the bands. Wood, itself, when kept saturated with water, has an almost indefinite life, old water mains in Philadelphia, New York, and Boston having been found perfectly sound after sixty or seventy years of use.

63. Vitrified Clay Pipe has been employed in a few places for conduits. It is cheap, indestructible, and when the joints are carefully made the leakage is very small. It is generally used under no pressure, but in one or two instances has been designed to carry considerable pressures.

64. Materials for Service Pipes. Service pipes, or pipes for conducting water to individual consumers, are made of a considerable variety of materials. Galvanized, tin-lined, lead-lined, and cement-lined iron pipe are widely used, but the most common is lead pipe. Lead pipe is practically indestructible, but rather expensive and heavy for high pressures. In some places it cannot be used with safety on account of the danger of lead poisoning. Certain waters only will attack lead to a sufficient extent to render its use dangerous, but, despite the study that has been put upon the subject, it is not yet fully known, without actual experiment, what effect various classes of waters will have.

Tin-lined pipe is now being used to a considerable extent. It is quite expensive, but the experience with it so far indicates that it may be very durable.

CONSTRUCTION OF CONDUITS.

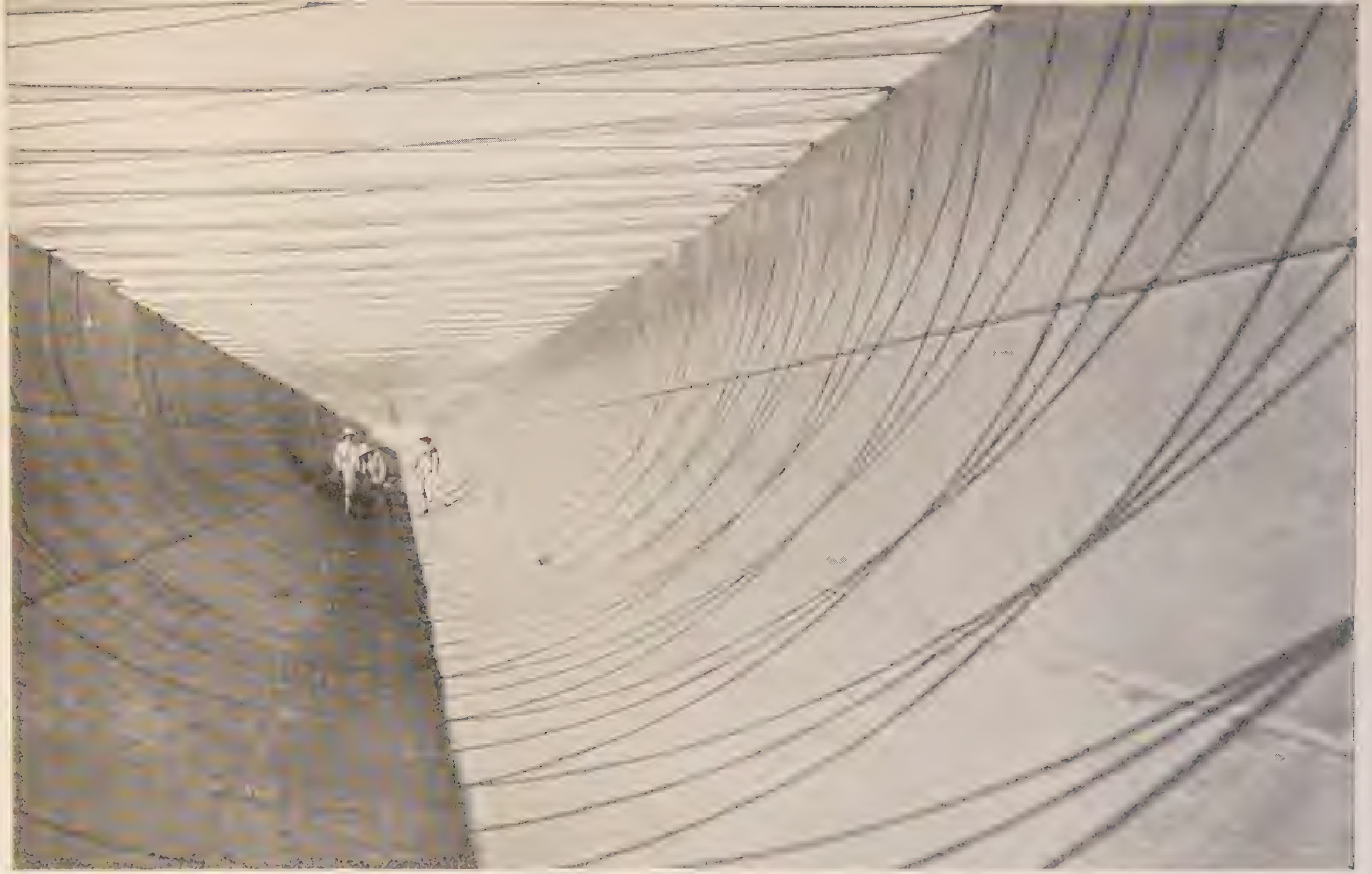
65. Classes of Conduits. Conduits are divided into two general classes: (1) those in which the water surface is free and the conduit therefore not under pressure, and (2) those flowing under pressure. To the first class belong open canals, flumes, aqueducts, and usually tunnels, and to the latter belong pipe lines of iron, steel, wood, or other material capable of resisting hydraulic pressure, and sometimes tunnels. Conduits of the first class must obviously be constructed with a slope equal to that designed for the water surface, or equal to the hydraulic gradient. This will be a very light and uniform slope, and such conduits will therefore often require in their construction long detours to avoid hills and valleys, or resort must be had to high bridges, embankments, cuttings, or tunnels. Conduits of the second class may be constructed at any elevation below the hydraulic grade line.

Long conduits usually include both masonry aqueducts and pipe lines, each class being used where most suitable. The former is used as a rule where the ground lies near or above the hydraulic grade line, and the latter where it lies below for any considerable distance. High and long aqueduct bridges are no longer built, a pressure conduit being substituted, which may follow the ground profile closely.

66. Canals. The open canal is not often used for conveying water for city use, but for irrigation purposes it is the common form of conduit. For the former purpose it has several objections, such as loss of water by percolation and evaporation, exposure of water to pollution from surface drainage and otherwise, and exposure to summer heat, which not only warms the water but promotes vegetable growth. However, where a canal can be constructed with little cutting or embankment, and where the material is nearly impervious, it may be the best form of construction.

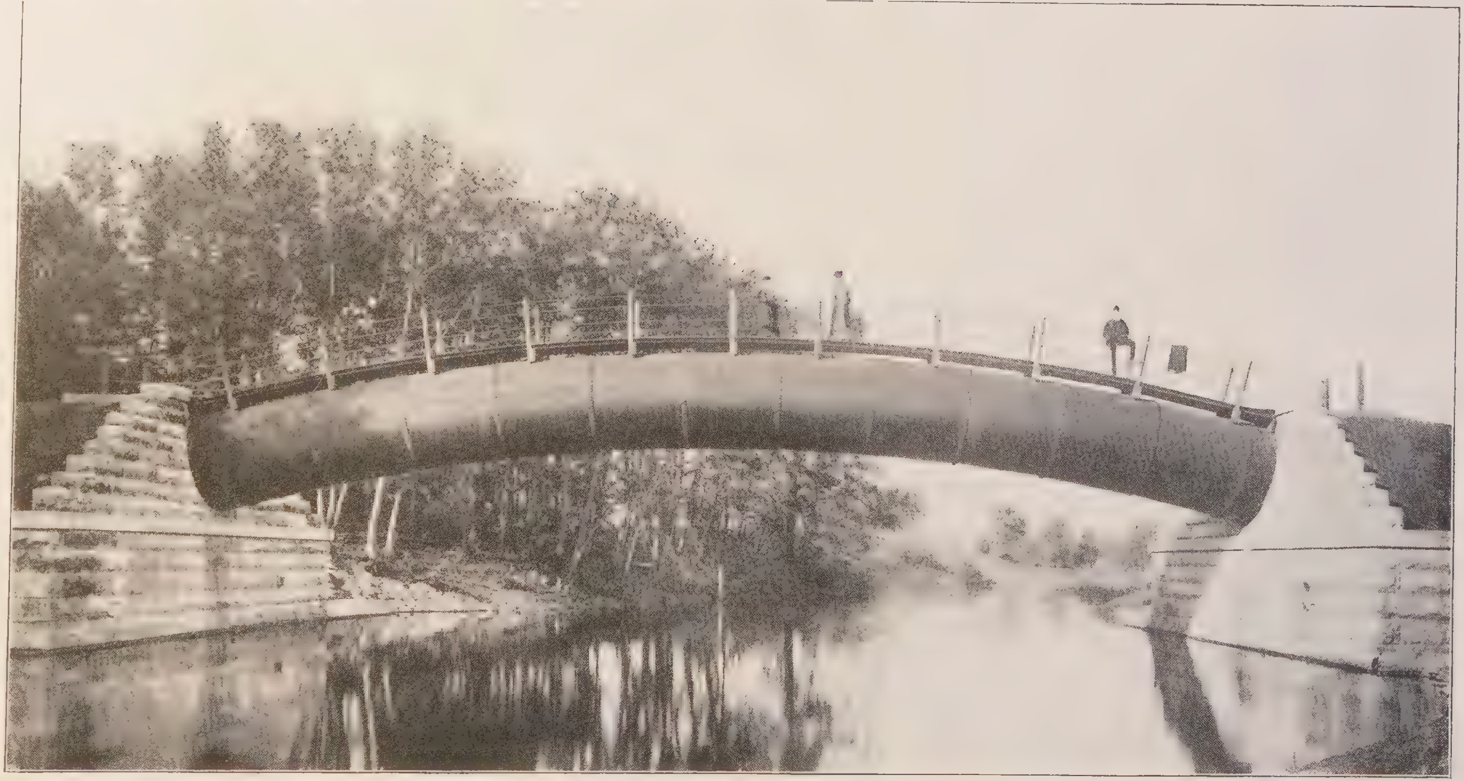
The allowable velocities for unprotected canals vary from about 1½ to 2 feet average velocity for light sandy soils, 2½ to 3 feet for ordinary firm soils, and 3 to 4 feet for hard clay and gravel. In rock or hardpan 5 to 6 feet may be allowed. A velocity of 2 to 3 feet per second is sufficient to prevent silt deposits and the growth of weeds.

The velocity and discharge for any given slope and cross-section is calculated from Kutter's formula as explained in Hydraulics. In using this formula the selection of a proper value of n is a matter of much uncertainty. For unlined channels it is usually taken at .020 to .025. If vegetation is allowed to accumulate in the canal, a large allowance must be made for increased resistance caused thereby. The cross-section of a canal is usually trapezoidal in form. Fig. 28



IRRIGATION CANAL BUILT OF STEEL

Designed to supply a parched region with water from the River Nile. Water is first pumped into a reservoir, then led through this canal one mile to the distributing channels. Built of steel plates riveted together, stiffened on the outside with T-irons and braced across the top with cross-angles. To allow for contraction and expansion, it consists of several sections connected by packed expansion-joints in masonry work.



PIPE-ARCH BRIDGE OVER SUDBURY RIVER, NEAR SAXONVILLE, MASS.

Part of the waterworks system of Boston. Width of span 80 feet, central part of arch rising 5½ feet above horizontal. Arch consists of double-riveted sections of steel pipe, ⅝ inch thick and 7½ feet in diameter, sustaining both its own weight and that of the water it contains. Stone abutment reinforced with a backing of 40 feet of solid concrete. A larger bridge built on same principle, supported by two arched mains, spans Rock Creek, between Washington and Georgetown, D. C.

shows a section built principally by embankment. Clay puddle is placed in the center of each embankment.

Side slopes in ordinary soils will vary from 1 to 1 for hard clay and gravel, to 3 to 1 or 4 to 1 for fine sand. The tops of the bank should be from 1 to 2 feet above the water line. If the soil is very porous, a lining of concrete or puddle may be necessary.

At sharp bends, and wherever the velocity exceeds the safe velocity for the material, some form of revetment is necessary. This may be merely a layer of gravel, or a paving laid dry or in cement, or a layer of concrete, according to the velocity of the water.

Waste weirs and sluice gates should be provided at intervals along the canal to prevent flooding and to permit of rapid emptying, but the flow in the canal is regulated for the most part by sluice gates at the head of the canal. These and other forms of canal gates are supported either by masonry walls or by timber framework.

Canals are carried across valleys on trestles or bridges, or, in the case of short crossings, on embankments with a culvert or arched bridge beneath. Under crossings are made by means of inverted siphons of pipe.

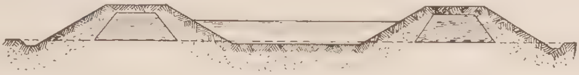


Fig. 28. Canal Section in Embankment.

67. Masonry Conduits. For conveying relatively large quantities of water over territory where the conduit can readily follow the hydraulic grade line, the masonry conduit is a preferable form of construction. If properly constructed it is very durable, requires little attention, and if the topography is favorable it is much cheaper than large pipe conduits of iron or steel. Masonry is unsuited to withstand tensile stresses, hence it is not used to convey water under pressure. Masonry conduits should not often be employed for cross-sections less than 10 or 15 square feet, for, unless the location be very favorable, their cost for such small sizes is likely to be greater than that of steel or iron pipes. The velocity should preferably be such as to prevent deposit of sediment, which requires $2\frac{1}{2}$ to 3 feet per second average rate; and for brick or concrete masonry it should not exceed 6 or 7 feet per second. Higher velocities may be allowed if stone masonry of hard material is employed, or if a lining of iron or steel is used. If sufficient head is available, a smaller conduit will result if the velocity is made as large as the material will stand without danger of excessive wear.

Kutter's formula is usually employed in these calculations. (See Hydraulics.) The value of n to be used will vary with the character of the masonry about as given in Hydraulics.

Brick is the most suitable material for linings, and is commonly used also for the entire arch crown. For the side walls and foundation, rubble masonry or concrete is employed. For places of great wear paving brick in cement is a good substitute for granite. Concrete is better suited than either stone or brick for irregular forms and especially for light sections. For small aqueducts a rectangular form has often been used, as in Fig. 29, the cover being of stones, slabs, or arches. For large sizes the horseshoe shape is better adapted, as shown in Fig. 30, which represents the form adopted for a large conduit for Boston.

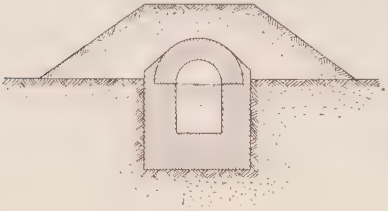


Fig. 29. Gallery, Vienna Waterworks.

The thickness of the arch is made about one-tenth to one-sixth the width of the opening, and of two, three, or four rings of brick, or a corresponding of concrete, depending on span and weight of covering. The arch is generally segmental in form. The invert, in compact ground, is made only thick enough to secure a firm and impervious bottom, two or three rings of brick, or a thin layer of concrete with brick lining, being usually employed. A timber foundation and sometimes piling may be required on soft soils. Settlement must be reduced to very low limits, or cracks and leakage will result. It is unnecessary to state that in work of this kind the masonry must be constructed with the most careful supervision. Concrete and stone masonry should be

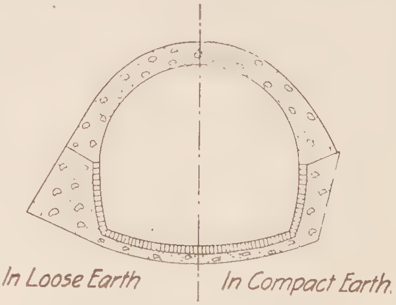


Fig. 30. Forms for Large Conduit.

given one or two finishing coats of thin neat cement to secure imperviousness, the last coat to be finished as smooth as practicable. If carefully done, and no settlement occurs, the leakage will be slight.

The aqueduct should be covered to a depth of 3 or 4 feet to prevent the formation of ice and to protect the masonry. Embankments should be given a slope of 1½ to 2 horizontal to 1 vertical according to the nature of the material. They should be trimmed to a rounded outline and then sodded.

Culverts for crossing small streams, and bridges for larger ones, are a part of the design. Some of the most monumental works of history are the bridges which have been built for carrying aqueducts. Large aqueduct bridges are now seldom constructed, pipe lines being substituted, but bridges of moderate size will often be the more economical design.

68. Pipe Lines. As to location, a pipe line must follow in general the variations of the ground surface, and such a location should be selected as will enable it to do so and at the same time give low pressures.

Where the total available head is fixed, the size required for any given capacity is readily determined by the table of the flow of water in pipes in Hydraulics. In case the water contains suspended matter, it is desirable to maintain a self-cleansing velocity of 2 to 2½ feet per second, otherwise the sediment must be blown out at frequent intervals. If the line is divided into sections by reservoirs or overflows, the size of each section is determined independently of the others.

If pumps are used to force the water through the pipe then the proper size depends on the relative cost of pipe, and of pumping against an increased head. A large pipe gives low friction head of the water and therefore saves in pumping expenses, but a large pipe is more expensive than a small one. In general it may be assumed that a proper size of pipe is one which calls for a velocity of flow of from 1½ to 2½ feet per second, the former value for pipes of 6 to 12 inches in diameter, and the latter for pipes of 3 or 4 feet in diameter.

69. Laying of Pipes. Trenches for water pipe are not usually deep enough to require much bracing or sheeting, the depth being ordinarily only sufficient to give the necessary covering. Deep trenches will, however, be required occasionally, as where the pipe line crosses a high ridge extending above the hydraulic gradient.

The laying of cast-iron pipe is usually begun at a valve or special. Small pipe up to 6 or 8 inches in diameter is easily handled without a derrick, the sections being lowered into the trench by two or three men. In laying, care should be taken to enter the pipe to its full depth and to see that there is sufficient joint space all around. If special strength is not required, this packing may nearly fill the space back of the enlargement or V-shaped space in the bell. The remaining space is filled with molten lead. In pouring the joint the lead is guided into the space by a jointer, commonly made of clay formed around a length of rope. This is placed about the pipe so as to press against the hub, except at the top, where an opening is made for pouring. Patent jointers are better for large pipe and difficult work. After pouring, the lead is loosened somewhat from the pipe by means of a chisel and set up by calking iron and hammer. To do good work there should be plenty of room around and under the pipe. In wet trenches and with small pipe, two or three sections may be joined before lowering. Riveted pipe should be connected up in as long sections as practicable before being transported to the trench, so that as much of the riveting may be done by power riveters as possible.

When placed in the trench the pipe should have an even bearing on firm soil or on blocking, and should be well supported while the joints are being riveted. The riveting is usually done by hand, but power riveters have been used in a few cases. After riveting, all field joints should be calked, and these and all other abraded places painted. Some re-calking may be needed after the pipe is tested.

In constructing the pipe system one of the most important points to settle is the depth at which the pipes should be laid. In warm climates a covering of 2 to 3 feet is sufficient. In cold climates the depth to be adopted is that which will be sufficient to prevent freezing. In a general way it may be stated that for a latitude of 40° the depth of cover should be 4 to 6 feet and for 45° should be 6 to 8 feet, the smaller depths being used east of the lakes and the greater depths for the country between the lakes and the Rocky Mountains. In sandy soil the depth should be greater than in clay.

70. Special Details. To enable a pipe line to be readily inspected and repaired, stop valves should be inserted at intervals of 1 or 2 miles, and especially at important depressions and summits.

Otherwise to empty and refill a long conduit would require several days. Valves of all kinds and designs are furnished by various special manufacturing concerns.

At every summit of a pipe line and at shut-off valves there should be placed an air valve to permit the escape of air on filling, the entrance of air on emptying, and frequently the escape of air which may gradually accumulate at summits. At all depressions, blow-off valves should be provided, the waste pipes from which should be led to a sewer, stream, or drainage channel. These valves need be only about one-third the size of the main pipe. Check valves should be introduced at points where a breakage would permit a large loss of water by backward flow, such as at the entrance to reservoirs, at the foot of long upward inclines, and in force mains just beyond the pumps. Safety valves, or pressure-relief valves, are occasionally used at the ends of long pipe lines or wherever water hammer is especially to be feared. They are simple disk valves opening outwards and held in place by springs which are adjusted to the water pressure.

The upper end of a gravity pipe line is usually enclosed in masonry and provided with a sluice gate or valve. At this point it is also desirable to have a weir or measuring sluice. The lower end of a pipe line usually terminates in a reservoir, where again valves are provided and where connections may also be made directly with the pipe system.

In crossing under other structures, such as railways, buildings, sewers, etc., special precautions should be taken to avoid all danger of future breakage.

Streams are crossed either on bridges, or by laying the pipe beneath the stream bed, or by the use of a subway.

In this country the common practice in crossing a stream is to lay a cast-iron or steel pipe below the stream bed, or else to employ a bridge crossing. Where no bridge already exists the former will ordinarily be the cheaper; and in many cases, as in navigable channels, a bridge could not be permitted. In other cases it may be cheaper to build a bridge especially for this purpose. At the angles at ends of bridge and submerged crossings special care is necessary to keep the pipe from separating at the joints. If the pipe line crosses an existing bridge, it will usually be convenient to support it beneath the flooring. Where a bridge is built for the purpose, no floor system is put in, but merely suitable straps or stirrups to support the pipe.

The amount of protection required to prevent freezing on bridges, or at other exposed places, depends upon the size of pipe, the amount of circulation during periods of minimum flow, the temperature of the air and the water, and upon the length of the exposed portion.

Small lines, especially distributing mains, require protection. This is usually furnished by placing the pipe in a wooden box and filling around it with some non-conducting substance, such as sawdust, mineral wool, asbestos, hair felt, and the like. A mixture of plaster of Paris and sawdust has been used with good results. Any packing to be effective should be kept dry. The packing is often arranged to give one or more dead air spaces around the pipe to aid in preventing radiation.

Various methods are employed in laying pipes beneath watercourses. In the case of small streams the usual method is to employ a cofferdam and lay the pipe as on dry land. Where the water cannot readily be excluded in this way the pipe must either be put together before lowering in place or must be laid by divers. Submerged pipe should, as a rule, be laid in a trench and carefully covered to prevent injury by waves, drift ice, boats, etc.

Various special details are used in submerged-pipe laying, such as the various forms of flexible joints to enable the pipe to conform to the grade of the trench, and special joints for easy connection where divers are employed. Submerged pipe should be thoroughly tested either in sections before laying, or better, after the line is completed, in which case compressed air can be used for the purpose. Leakage of air will be indicated by the appearance of bubbles, and the imperfect joints can then be calked by divers. The various methods of laying submerged pipe will now be described together with some of the special details used in this work.

(1) Where the stream is shallow, a common method of laying is first to connect the entire pipe, or large sections of it, on platforms extending across the stream, and to lower the portion so connected by means of screws. Ordinary joints can usually be employed and the pipe put together to fit the profile of the trench. Pipes can very conveniently be laid in this way from the ice during winter.

Two cases of this method of laying will be briefly noted. At Cedar Rapids, Ia., 600 feet of 16-inch pipe was laid in this way in a depth of $2\frac{1}{2}$ feet of water. A trench 2 feet deep was first excavated, and framed trestle bents set up 12 feet apart. A barge was then run between the legs of the trestles, the pipe put together on the barge and then slung by straps fastened to $1\frac{1}{4}$-inch threaded rods suspended from the trestles. When the entire pipe line was connected, it was all lowered together, electric-bell signals being used to secure simultaneous action among the several men stationed at the screws. The cost of laying was $1.25 per foot.

(2) Instead of connecting the entire pipe line and lowering all together, it may be lowered in sections by the aid of flexible joints, each section consisting of several lengths of pipe connected in the usual manner. The pipe can thus be laid and lowered from a short piece of trestle or from a barge. This method is especially suitable for deep water where trestles cannot readily be used.

(3) Many lines of submerged pipe have been laid by joining several lengths on shore, towing them into position, sinking them and connecting them by divers. This method is especially applicable for large pipe lines. It has been used for large intakes at Syracuse and at Milwaukee; also at Galveston, Nashville, Boston, and many other places.

71. Cost of Pipe Lines. The cost of pipe lines will vary greatly according to the cost of the material used. This element can readily be ascertained at any time by reference to current price lists, and the item of transportation can also be quite readily determined. Cast-iron pipes laid under average conditions will cost approximately as follows, assuming the pipe itself to cost $1\frac{1}{2}$ cents per lb

Size of pipe.	Cost per foot.
4 inch	$.50
6 "	.70
8 "	1.00
10 "	1.30
12 "	1.70
16 "	2.50
20 "	3.50
24 "	4.50

THE DISTRIBUTING SYSTEM.

Distribution Reservoirs.

72. Use. The rate at which water is actually used is not at all uniform, as fully pointed out in section 6. It varies from day to day according to the season, from hour to hour according to the time of day, and at times of large fires the rate will be greatly increased. If all parts of a system were to be designed of a capacity equal to the greatest possible rate of demand the cost would frequently be prohibitive, and in most cases it would not be the most economical plan. It will usually be more economical to store up a quantity of water in a small reservoir or elevated tank which may be drawn upon when the demand is excessive and thus relieve to some extent a part of the system.

For example, where the water is brought from the source through a long conduit, a distributing or equalizing reservoir will enable the conduit to be operated at a comparatively uniform rate and hence to be made of minimum size. Likewise such a reservoir will make it possible to reduce the capacity of pumps, or filters, or other similar works, and to operate them more uniformly and economically; or in the case of small works to operate the pumps at full capacity for a portion of the day only. In the case of a ground-water supply, a small reservoir will greatly increase the capacity of the source by making the demand more uniform. Again, in a large distributing system, several reservoirs placed at different points will effect considerable economy in the size of the pipe system. As a measure of safety against the interruption of the supply from accidents to conduit or machinery, distributing reservoirs are of great value.

In discussing forms of construction, reservoirs may be classified, according to the material employed, into (1) earthen reservoirs, (2) masonry reservoirs, (3) iron or steel reservoirs, and (4) wooden reservoirs. The first two kinds can conveniently be considered together, as the two materials are very often combined in the same structure. The last two will also be treated under the general title of standpipes and tanks.

When the reservoir does not need to be elevated above the natural surface, the most economical form, and the usual one for large capacities, is the open reservoir with earthen embankments. Such reservoirs are usually built with masonry walls, and covers partly in excavation and partly above the surface. If a reservoir requires to be considerably elevated, a steel standpipe or a tank of wood or steel is usually employed.

73. Capacity. Where it is possible to construct an inexpensive open reservoir at a suitable elevation and in a good location, it should be given a capacity of several days' supply. In practice the capacity of such reservoirs varies from 2 or 3 days' supply up to 8 or 10 days, and occasionally more.

Where, owing to the character of the topography, it becomes necessary to artificially elevate a reservoir in the form of a standpipe or elevated tank, the expense of construction becomes so great that the economical capacity is usually less than that mentioned under (1). The best capacity in this case depends much upon the size of the city. For large cities it is hardly practicable to provide much storage by means of artificially elevated reservoirs, the small standpipes which are often used in such cases serving merely to equalize the action of the pumps.

In small cities (up to a population of 50,000 or more) it is desirable to provide a small storage even at considerable cost, as a measure of safety and economy. The fire rate is here the principal consideration, and the minimum capacity should be such as to provide water at the maximum fire rate for a sufficient length of time to enable the pumping station to respond with ease and certainty. This is ordinarily taken as about one hour. Beyond this it will usually be desirable to add to the capacity enough to equalize the ordinary flow over several hours of the day, or, in the case of small works, to enable the pumping to be done by operating a part to the day only.

In general the best size of elevated tank will range from about 75,000 gallons for very small works up to 200,000 or 300,000 gallons for towns of the size mentioned above.

74. Location and Arrangement. The location of an elevated reservoir is governed in the first place by the topography, and the choice of location is therefore often very limited. In general a distributing reservoir should be located as centrally as possible with respect to the district to be served, as this will insure the most uniform and the highest pressures and will give the smallest size of main and branches.

In a gravity system the conduit is terminated at a reservoir, and if this reservoir is centrally located a longer conduit will be required than if it be placed near one side of the system. A proper balance must be struck between the two extremes. In a pumping system the pumps are usually located near one side of the city, and the reservoir is placed either in the vicinity of the pumps or at a more remote point in the system. In the first case all the water is usually passed through the reservoir, and the action of the pumps is very steady and uniform. In the second case a main usually leads to the reservoir from some point of the distributing system. The pumps force water directly into the system, and the reservoir takes only the surplus at times of low consumption and distributes it at times of high consumption. Certain portions of the area are thus served direct, and others are served from the reservoir. With this arrangement a more uniform pressure will be maintained in the mains, but the operation of the pumps will not be as uniform.

The proper elevation of a reservoir depends on the required pressure in the mains, a subject fully discussed in section 89.

EARTHEN AND MASONRY RESERVOIRS.

75. Form and Proportion. Earthen reservoirs are usually constructed partly by excavation and partly by the building up of embankments. If masonry walls are used in place of embankments, or as interior linings, the reservoir may be called a masonry reservoir. For single reservoirs the form most economical of material is the circular, but for large reservoirs the rectangular form is more convenient to construct and requires less land area. In practice the depths vary from 12 to 18 feet, for small covered reservoirs holding one million gallons or less, to 25, 30, or 35 feet, for open reservoirs holding 50 or 100 millions, depending upon local circumstances.

76. Construction. The construction of the embankment is based on the same principles as discussed in section 47, but the conditions are somewhat different from those obtaining with impounding reservoirs. In this case the foundation is frequently pervious and the embankment cannot be connected with an impervious stratum below. Under such conditions it is necessary to construct a water-tight lining over the entire area, and to carefully connect it with the water-tight portion of the embankment. Where a lining is not necessary to secure imperviousness, one is usually put in to facilitate the cleaning of the reservoir.

According to circumstances the entire embankment may be impervious, or imperviousness may be secured by a puddle or concrete core, or by a layer of puddle placed near the face. Fig. 31 shows a puddle being placed near the face and Fig. 32 shows a puddle core connected to the puddle lining of the bottom.

Imperviousness is usually secured in large masonry reservoirs by a layer of puddle placed back of the wall and thoroughly rammed, and the bottom lining is treated in a similar way. In small reservoirs more reliance is placed upon impervious masonry, made so by an asphalt coating, or, more commonly, by a coat of Portland-cement mortar, neat or 1 to 1, which it is well to finish by a brush coat of neat cement. The latter method is more likely to be satisfactory with covered reservoirs, where the temperature changes are small, than with open reservoirs.

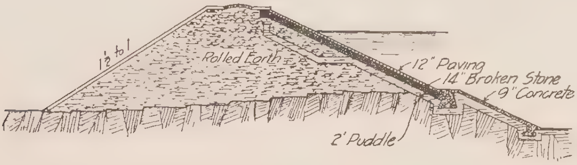


Fig. 31. Section of Reservoir Embankment, Pittsburg.

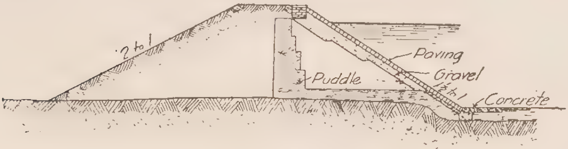


Fig. 32. Section of Reservoir Embankment, Brooklyn.

While it is comparatively easy to secure imperviousness at the start by the use of cement, it is difficult to prevent the formation of slight cracks. These permit the water to find its way into the surrounding soil, and when the reservoir is quickly emptied this water exerts a back pressure on the walls and an upward pressure on the floor. The foundation for the walls should be broad enough to reduce settlement to very small limits, and as further precaution against cracks the floor lining should be constructed after the walls are complete, but should be thoroughly bonded thereto. Junctions between floors and walls are preferably made curved.

The most common form of lining consists of about $1\frac{1}{2}$ to 2 feet of puddle protected by a layer of concrete, brick, or stone paving, or sometimes only by gravel. On the slopes the concrete is usually covered with paving or replaced entirely by it, experience showing that unprotected concrete is apt to be injured by ice. A layer of paving brick laid in cement makes a good finish for a concrete lining which is to be frequently exposed. Concrete can be made impervious by plastering with cement mortar, neat or 1 to 1, but where it extends over large areas, cracks will form, due to temperature changes and to settlement of embankments. To minimize this difficulty, concrete may be laid in blocks, with asphalt joints between.

If ground water is met with, which is under considerable pressure, it will be necessary, in order to avoid rupture of the floor, to drain the soil beneath the lining. In some cases the ground water has been permitted to enter the reservoir, when its head exceeds that in the reservoir, through flap valves which will close when the difference of head is in the reverse direction. Drainage of the soil beneath the lining should be done with great caution, and especial care taken to surround all drains with gravel and sand so graded in fineness as to effectually prevent the washing out of any of the material. Seepage water is also sometimes taken care of by means of drains.

Asphalt has for some time been extensively used for reservoir linings on the Pacific coast, and recently its use has become quite general. Compared to concrete it has the advantages of elasticity and greater imperviousness, both of which are of great importance in this connection. Another advantage in many cases is its cheapness. Its chief disadvantage is the effect of the sun in rendering it more or less plastic and liable to creep if used on steep slopes. Its durability in water is also not fully determined. Great care and expert knowledge are required in determining the proper proportions of the various ingredients to give good results.

When the earth is firm and compact, asphalt linings can be placed directly upon it, and have frequently been so placed. Considerable settlement has in some cases taken place without cracking the lining, but this cannot, of course, be relied upon.

Reservoirs with masonry walls occupy less space than earthen reservoirs, but are more expensive to construct. They are, however, often the best form for small reservoirs where space is limited, and are a suitable form in case covers are required.

When the reservoir is excavated in firm earth or is backed by a well-compacted embankment, the earth serves to support the walls against water pressure. They must then be designed to sustain the earth pressure with reservoir empty. By adopting the circular form the masonry will resist largely by compression as a ring, and the dimensions can be considerably reduced below those required for a wall resisting by gravity alone. Several small circular reservoirs have been built of diameters of 50 to 75 feet, with walls from 16 to 22 inches in thickness.

The masonry may be of rubble, concrete, or brick, according to circumstances. If exposed, a lining of paving brick makes an excellent finish. It is needless to say that in all work of this character the greatest care should be taken to secure the best workmanship, particularly in the mixing and laying of concrete and the thorough filling of masonry joints with mortar, essentially as in dam construction.

77. Inlet Pipes and Valves. Distributing reservoirs are usually provided with separate inlet and outlet pipes, located preferably on different sides of the reservoir in order to promote circulation of the water. In earthen reservoirs these are constructed in the same manner as described in section 48. A by-pass should be provided to enable the reservoir to be cut out at any time. Where the reservoir serves merely as an equalizing reservoir, receiving only the surplus water from the distributing system, a single pipe will serve for both inlet and outlet.

In open masonry reservoirs gate chambers are conveniently built in connection with the reservoir wall. In covered reservoirs they are usually omitted, the valves being placed within the reservoir and operated from a suitable platform or from the oustide.

78. Covered Reservoirs. Ground waters should be stored in covered reservoirs, for the reason that such waters usually contain sufficient quantities of plant food to promote a luxuriant growth of vegetable organisms unless the light be excluded. Many cases have arisen of bad tastes and odors due to this cause which have been entirely removed by covering the reservoir. Filtered surface waters should also as a rule be stored in covered reservoirs, since by the process of filtration they are rendered somewhat similar in nature to ground waters. Where reservoirs are located in the densely populated portions of cities, covers are also advisable, in order to exclude soot and dust.

Covers are usually made of masonry, but wood has been used in a number of cases. It is much cheaper than masonry, but is much less durable and does not keep the water as cool in summer or wholly prevent freezing in winter.

A wooden cover for a large area may consist simply in a horizontal floor of boards, supported by a system of joists and girders resting on a series of wooden posts. For small areas the covers can readily be made sloping, and this is a preferable arrangement. Covers for small circular reservoirs and large wells are conveniently made conical, with the rafters resting against the wall or supported on light trusses.

Masonry covers consist usually of segmental or elliptical masonry arches supported by small brick piers; or, for very small reservoirs, a dome may be used. Above the arches, about 2 feet of earth is placed to prevent extreme variations of temperature and to protect the masonry, and embankments are constructed against the side walls to meet the covering above. The piers are spaced from 10 to 15 feet apart, and are made from 1 to 2 feet square in cross-section, depending upon the span and weight of filling.

Piers are usually made about 18 inches square of brick and spaced about 12 to 15 feet apart. Concrete is now generally used for the roof, being made in the form of groined arches of about 3 feet rise and 6 inches thick at the crown. Fig. 33 shows the interior of such a reservoir used as a filter.

79. Cost. The cost of reservoirs varies, of course, greatly according to local conditions, kind of reservoir and capacity. According to the capacity the cost per unit will be less the larger the reservoir. The actual cost of a large open reservoir varies from $3 to $5 per 1,000 gallons capacity. Covered masonry reservoirs will cost usually from $10 to $15 per 1,000 gallons capacity.

STANDPIPES AND ELEVATED TANKS.

80. Where a reservoir requires to be artificially elevated it is usually built as a standpipe—a tall slim tank resting on the ground—or as an elevated tank of steel or wood, supported by a tower of steel, wood, or masonry. Such an elevated reservoir may or may not be enclosed in a covering of masonry or wood, according to the necessities of the case and the notions of the designer.

Reservoirs of this type are relatively so expensive that a minimum amount of storage capacity is usually provided. As shown in section 72, they may be used in small towns to enable the pumps to be more economically operated, or in larger towns to provide for fire consumption for an hour or so. The capacities of standpipes and tanks range ordinarily from 50,000 gallons up to a maximum of about 1,500,000 gallons for small villages and cities up to 30,000 population or more. The useful capacity of a standpipe is only that part of the volume which is at a sufficient elevation to give the required pressure. All water below this level acts merely as a support for the portion above. There should therefore first be deter-

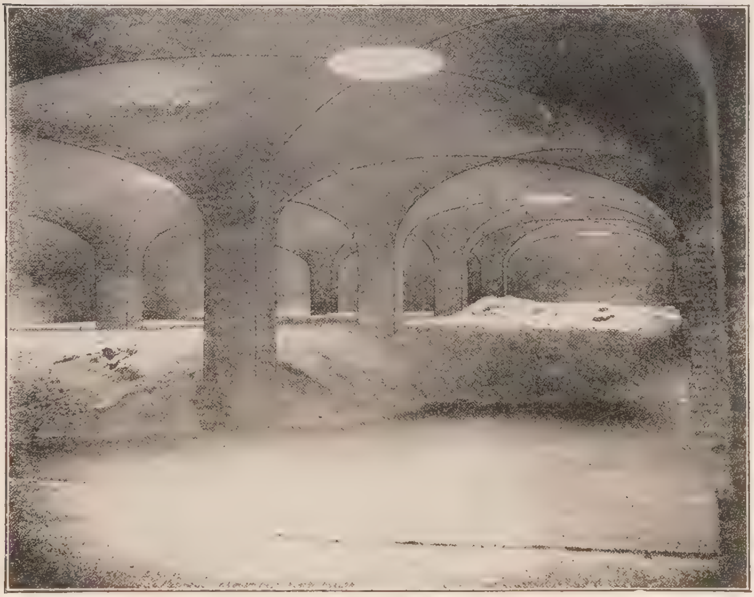


Fig. 33. Covered Filter.

mined the lowest useful level of the water, and the pipe should then be made of the desired capacity above this plane.

81. Location. For storage purposes only, the location would be the same as that for any reservoir. To reduce the cost it is, however, desirable to place the tank on the highest ground available if it be within a reasonable distance. Too great distances will be undesirable on account of the cost of mains and the loss of head caused by a long line of pipe.

82. Design of the Standpipe. The chief elements in the design of a standpipe are the thickness of plates, the riveting, the foundation and anchorage and the pipe details. The forces to be considered in the design of a standpipe are the water pressure, the wind pressure, the weight of the pipe, and the action of ice. In what follows let h = distance in feet of any point of the pipe from the top, d = diameter of pipe in feet, r = radius in feet, and t = thickness of shell in inches at the given point.

From equations in Hydraulics we find that the water pressure causes a bursting stress per vertical lineal *inch* of pipe equal to

$$S = \frac{62.5hd}{2 \times 12} = 2.6hd \qquad \textbf{(9)}$$

The stress per square inch of metal is

$$s = \frac{2.6hd}{t} \qquad \textbf{(10)}$$

This is the only stress that need be considered in determining the plate thickness, as the effects of wind and weight are much smaller than this and cause a stress in a *vertical* direction.

The safe tensile stress on net section of metal, where but little ice is likely to form, may be taken at about 15,000 pounds per square inch. Where thick ice is to be expected the working stress should be reduced to 12,000 or even 10,000 pounds, to provide for the unknown ice stresses. The vertical joints will usually be so designed as to have an efficiency of 60 to 70 per cent. If a = safe stress on net section and e = efficiency, then by equation 10 the required thickness to resist the water pressure will be

$$t = \frac{2.6hd}{s} = \frac{2.6hd}{ae} \qquad \textbf{(11)}$$

or, if $a = 12{,}000$ and $e = \frac{2}{3}$, then, approximately,

$$t = \frac{2.6hd}{8{,}000} = .000325hd \qquad (12)$$

The thickness near the top should not be less than $\frac{1}{4}$ inch, or for very large pipes, $\frac{5}{16}$ inch. Plates thicker than 1 inch or $1\frac{1}{8}$ inches should be avoided.

The plates forming a standpipe are usually of such a width as to build 5 feet of pipe, and are from 8 to 10 feet long. Each course is preferably made cylindrical, and alternately an "inside" and an "outside" course.

The riveting of the vertical seams is the most important part of the construction, as this determines the strength and economy of the standpipe. Lap joints are most commonly used, but for thickness exceeding $\frac{1}{2}$ inch, double-butt strap joints are much preferable and are stronger.

Table No. 12 gives suitable proportions for riveted joints, together with their approximate efficiencies or ratio of strength of joint to strength of plate.

TABLE 12.

Proportions for Riveted Joints for Standpipes.

Kind of Joint on Vertical Seams.	Thickness of plate.	Diameter of rivet.	Pitch of rivets. Centre to centre.	Distance between Pitch-lines.	Distance of Pitch-line from edge of plate.	Efficiency, per cwt.
	Inch.	Inches.	Inches.	Inches	Inches.	
Single-riveted lap	$\frac{1}{4}$	$\frac{5}{8}$	$1\frac{5}{8}$	.....	1	50
" " "	$\frac{5}{16}$	$\frac{5}{8}$	$1\frac{3}{4}$	.....	$1\frac{1}{8}$	
Double " "	$\frac{5}{16}$	$\frac{5}{8}$	$2\frac{5}{8}$	$2\frac{1}{8}$	$1\frac{1}{8}$	60
" " "	$\frac{3}{8}$	$\frac{3}{4}$	$2\frac{5}{8}$	$2\frac{1}{8}$	$1\frac{1}{4}$	
" " "	$\frac{7}{16}$	$\frac{3}{4}$	$2\frac{3}{4}$	$2\frac{1}{4}$	$1\frac{3}{8}$	
" " "	$\frac{1}{2}$	$\frac{7}{8}$	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{1}{2}$	
" " butt	$\frac{9}{16}$	$\frac{7}{8}$	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{5}{8}$	70
" " "	$\frac{5}{8}$	$\frac{7}{8}$	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{5}{8}$	
" " "	$\frac{1}{16}$	1	$2\frac{7}{8}$	$2\frac{3}{8}$	$1\frac{3}{4}$	
" " "	$\frac{3}{4}$	1	3	$2\frac{1}{2}$	$1\frac{3}{4}$	
" " "	$\frac{13}{16}$	1	3	$2\frac{1}{2}$	$1\frac{7}{8}$	
" " "	$\frac{7}{8}$	$1\frac{1}{8}$	3	$2\frac{1}{2}$	2	
" " "	$\frac{15}{16}$	$1\frac{1}{8}$	3	$2\frac{1}{2}$	2	
" " "	1	$1\frac{1}{4}$	$3\frac{1}{8}$	$2\frac{5}{8}$	$2\frac{1}{8}$	
Triple " "	1	$1\frac{1}{8}$	4	3	$2\frac{1}{8}$	75

Horizontal joints are made single-riveted lap joints, with rivet spacing of about three diameters. All seams should be thoroughly calked with a round-nosed calking tool, and any leaky seams which may exist when the pipe is filled should be recalked. The bottom is made of plates riveted up with circular and radial joints, the former being made lap joints and the latter butt joints. The thickness need be only enough to permit of good calking and to be durable,—about $\frac{1}{2}$ inch. This bottom plate is preferably connected to the side plates by means of a heavy angle on the outside, or one on both outside and inside the tank. The foundation should be made monolithic and sufficiently broad to give such low pressures on the soil that there will be practically no settlement. Failures have occurred due to poor work in this respect. Wind pressures should be carefully considered. Concrete is a very suitable material for foundation purposes.

Standpipes must be anchored to the foundation to prevent being overturned by the wind. The wind pressure is usually taken at 40 to 50 pounds per square foot on one-half the vertical projection of the tank. At the higher value the overturning moment in foot pounds at a distance h below the top is

$$M = 50 \times \frac{dh}{2} \times \frac{h}{2} = 12.5dh^2 \qquad \textbf{(13)}$$

This movement causes an uplift on the leeward side for each inch along the circumference of the pipe of

$$S^* = 1.33 \frac{h^2}{d} \text{ pounds.} \qquad \textbf{(14)}$$

Then if anchor bolts are placed p inches apart around the bottom of the tank the stress in each bolt will be

$$S \times p = 2.33 \frac{h^2}{d} \times p \qquad \textbf{(15)}$$

If numerous bolts are used, their size need not be great, and they may be put through the exterior bottom angle iron and the latter double-riveted to the pipe. If arranged in this way, they should be numerous enough so that the stress in one bolt is not greater than can be transmitted to the lower plates by four or five rivets, which will limit the size of bolts to about 1¾ times the diameter of the lower rivets. By spacing the bolts sufficiently close this arrangement may be followed in almost any case. If this method gives a large number of bolts, it will be simpler to use fewer and larger bolts, in which case they should be fastened to the standpipe by long vertical pieces of angles, and the bolts placed close to the pipe as shown in Fig. 34. The number of bolts should not be less than six in any case. Anchor bolts should extend well into the masonry and be fastened to anchor plates embedded therein.

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*The derivation of this equation comes from the formula $S = \frac{Mc}{I}$ of Mechanics in which M is the moment of inertia of the standpipe shell. The process of derivation cannot well be entered upon here.

Besides the overturning effect of the wind there is to be considered the collapsing effect on the empty pipe, especially near the top where the plates are thin. This cannot readily be computed, but must be provided for by an ample margin of strength at the top of the standpipe.

The effect of ice action is a very serious matter in unprotected standpipes, but is very difficult to calculate or provide for. The stresses caused by ice action can only be provided for by the use of a good quality of soft steel which will allow of deformation without injury, and by the use of a large factor of safety. It may well be questioned, in view of the uncertainties of the case, if all metal tanks built in cold climates should not be encased in masonry or wood. The importance of this matter is attested by the many accidents traceable to the action of ice.

The material used for standpipes should be soft, open-hearth steel, of a tensile strength of about 54,000 to 62,000 pounds per square inch. The best practice now calls for a grade corresponding to flange steel, with phosphorus limit of about .06 per cent, an elongation of 22 to 25 per cent, reduction of area of 50 per cent, and flat bending tests, both cold and after heating and quenching.

83. Pipes and Valves. Usually a single pipe serves both as inlet and outlet. This passes through an arched opening in the foundation, turns upwards and enters the standpipe at the bottom, and extends into it a foot or two. A lead joint is usually made in a bell casting riveted to the bottom of the pipe as shown in Fig. 34. A drain-pipe through which the tank may be drained or flushed should also be provided.

High-water electric alarms are advisable if the pipe be at some distance from the pumping station. The pressure indicated at the station is not a certain guide if branch mains are led off at intermediate points. For encased pipes or tanks a simple float, arranged to close an electric circuit, may be used. For exposed pipes, ice is likely to interfere, and in this case a pressure gauge placed in a vault and connected to the standpipe can be arranged to give an alarm at any desired pressure.

84. Other Details. The top should be stiffened against collapse by a heavy angle-iron, not less than 3 × 5 inches, and two such angles should be used for large pipes. The effect of the wind on an empty pipe is not only to cause a pressure on the outside, but to create a partial vacuum on the inside near the top. Several failures have occurred from lack of strength at this point.

It is not customary to roof standpipes, and for a tall slim pipe a roof would be of little use and no improvement to its appearance. With large, low pipes a conical roof of curved profile may well be adopted. It affords considerable protection and improves the appearance of the structure. It is usually made of sheet iron or copper, supported on light angle-iron ribs or a framework.

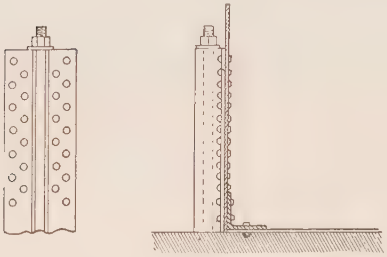


Fig. 34. Arrangement of Rivets.

A ladder should be built on the outside of the pipe, but none on the inside; and in general there should be no obstructions on the inside where ice is likely to form to any extent.

Standpipes should be well painted inside and out. For the interior, asphalt is probably the best material to use. After painting the interior, the pipe should be filled to detect leaks before the outside is coated.

A standpipe is often surrounded with a masonry shell in order to furnish protection from cold, or to improve the appearance of the structure. This shell may be of stone or brick, and is usually built enough larger than the pipe to permit of a stairway in the space between. The walls are usually made from 2½ to 4 feet thick at the bottom and 1½ to 2 feet at the top.

Encased pipes must be provided with overflows, which may be built either inside or outside the main pipe. For this type of structure, roofs are quite necessary; and should be carefully proportioned with respect to appearance. The masonry offers considerable opportunity for architectural treatment, and this feature should be referred to a competent architect.

85. Elevated Tanks. If the lower portion of the water in a standpipe is at too low an elevation for useful pressure, its only office is to furnish support to the useful part above. Where this useless zone is of any considerable depth the support can be more cheaply furnished by a steel trestle. Besides being cheaper, a tank is much less objectionable in appearance than a standpipe, and experience indicates that trouble from ice is less likely to occur. For roofed tanks a height equal to the diameter would not be far from the most economical proportions, but a height somewhat greater than this will usually look better.

The bottom of a tank supported on an iron trestle is usually made hemispherical, as this requires no support except at the outside edge where the legs of the tower are located. The thickness of side plates is the same as for standpipes, and the details are similar. If the bottom is hemispherical the stresses therein will be one-half those in the lowest side course of plates.

The tower consists of a steel trestle of four to eight legs. The material for this may be medium steel, and comparatively high working stresses may be used in its design, since the stresses are all dead- and wind-load stresses. Four legs are the smallest practicable number, but for tanks of large diameters the use of only four legs brings very heavy local stresses on the tank at the points of connection. Six or eight is a better number and presents a better appearance, but is more expensive. The stresses in the various parts of the tower and the design of the details belong to the domain of structural engineering and cannot be elaborated here. Suggestions for the upper column connections, the anchorage and the roof are given in Fig. 35.

Each column must be well anchored to the foundation, with a strength of anchorage equal to the maximum uplift due to wind acting on empty tank. The foundation should be rigid, and large and heavy enough to serve as anchorage and to give only safe pressures on the ground. There should be practically no settlement, as any unequal settlement will greatly change the stresses in the tower.

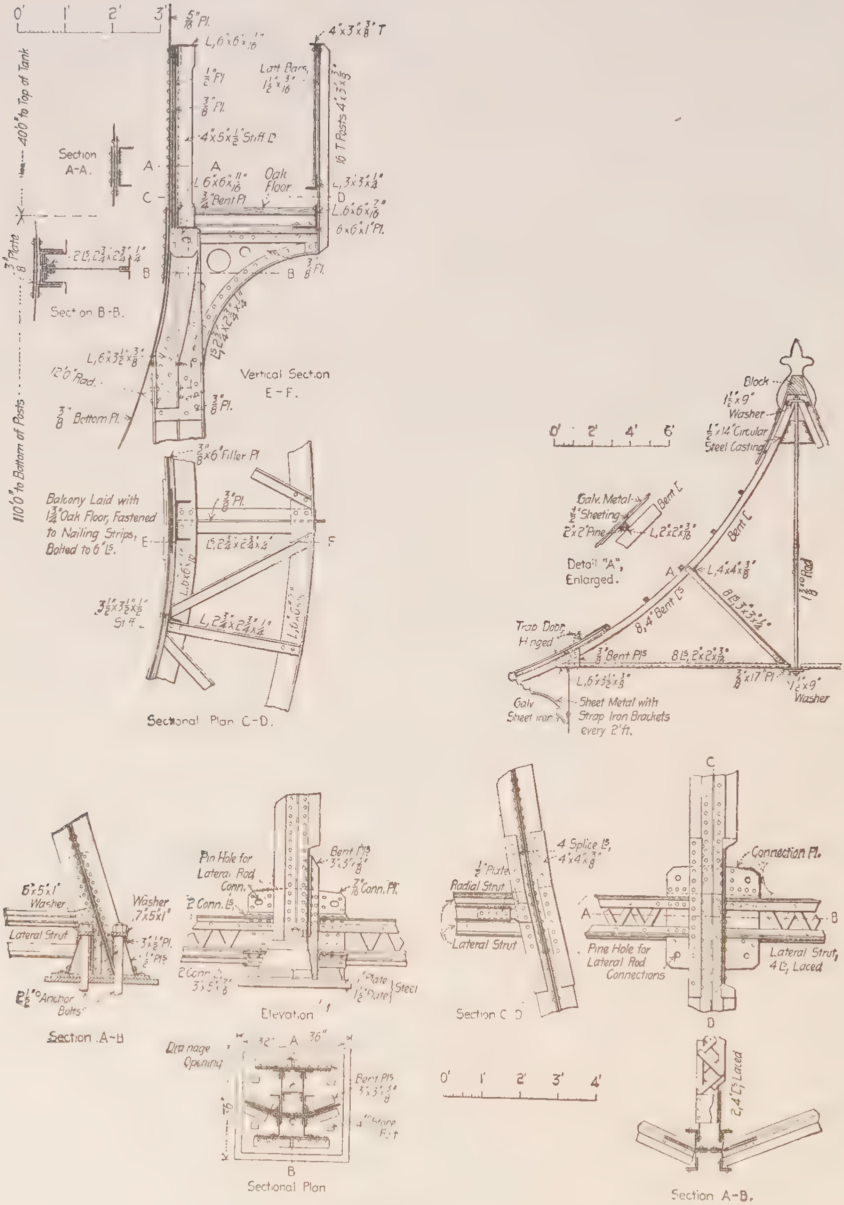


Fig. 35. Details of Elevated Tank, Ames, Ia.

The inlet pipe is usually made to enter the tank at the center of the bottom, and should be provided with an expansion joint. In cold climates the pipe must be protected by a frost casing, which is usually a simple wooden box with one or more air spaces and a packing of some non-conductive material. If the tank is encased, it will be necessary to provide an overflow pipe.

86. Wooden Tanks. Elevated tanks of wood are frequently used where low first cost is an essential element and the quantity to be stored does not exceed 50,000 to 75,000 gallons. Wooden tanks are cheap, and if well built will last fifteen or twenty years. The staves should be of good clear material and should be dressed to proper curvature on the outside. Hoops should be relatively thick to resist corrosion, and should be thoroughly coated with asphalt or other protective coating, before being put in place. Lugs and fastenings are a source of weakness. They should be carefully designed and of ample strength. The support of the floors must also be well looked after. The chief source of trouble with wooden tanks is in the weakening of the hoops by rusting from the inside.

Several failures of wooden tanks have occurred by the sudden bursting of the hoops, and it is questionable policy to construct such tanks where their failure is likely to endanger life, as it is quite certain that they will not be regularly inspected as they should be.

87. Storage Under Compressed Air. In small works, air chambers or their equivalent may be used to provide a considerable storage of water and thus avoid the use of standpipes or elevated tanks. In the design of such storage tanks the larger the proportion of air space the less will be the variation in water pressure as the tank is emptied. If V = volume of tank, and v = maximum volume of water stored, then $V - v$ = minimum volume of air. If the pressure, when containing the maximum volume of water, be P, then when the tank is just empty the pressure is $p = P\left(1 - \frac{v}{V}\right)$. Thus if $\frac{v}{V} = \frac{1}{3}$, then $p = \frac{2}{3}P$, and the variation in pressure is one-third the maximum. The less the desired variation in pressure the greater must be the tank capacity for a given water capacity. The air is maintained in the tank by occasionally admitting a little air into the pump.

A system of pressure storage having several advantages over that just described is the Acme Company's system, based on patents of Wm. E. Wortham and Oscar Darling. In this system the air is stored in a separate tank at a higher pressure than is ordinarily kept in the water tank. By reducing valves in the connecting pipes, the pressure on the water may be maintained constant, or may be increased in case of fire up to the pressure in the air tank. Air compressors must be used here to keep up the air supply. A number of plants of this kind have been installed. The use of a pressure storage system avoids all trouble from ice, and for very small quantities is cheaper than an elevated tank. A storage tank can also be located at the pumping station and the pressure easily controlled. For large quantities the system would be very expensive.

THE DISTRIBUTING PIPE SYSTEM.

88. General Requirements. A distributing system should be so designed that it will be able to supply adequate quantities of water to all consumers, and that this will be accomplished with economy and with reasonable security against interruption. With respect to the design of this part of a waterworks system, the uses of water naturally fall into two very distinct classes: (1) the ordinary, everyday use for domestic, commercial, and public purposes; and (2) the use for fire extinguishment. In the former case the consumption is relatively uniform over different portions of the city, and is also well distributed over many hours of the day; in the latter case the rate is likely to be extremely high for a very short period of time, but this excessive use of water will usually be confined to a comparatively small area. To supply water in the former case requires the wide distribution of moderate quantities, while in the latter case the problem is rather the concentration of large volumes within a narrow district, which district may be situated at any point in the system.

To supply water to all consumers requires that a pipe be laid in each street, except in those cases where the cross streets are not built upon. In the outlying districts, pipes are laid in those streets where the density of the population warrants it, according to the judgment of the management, but much difference in policy exists in respect to the matter of extensions. The distributing system includes, besides the pipes, the fire hydrants, service connections, valves, fountains, watering troughs, meters, and occasionally other details.

89. The Pressure Required. For ordinary service the pressure at any point should be sufficient to supply water at a reasonable rate in the upper stories of houses and factories, and in business blocks of ordinary height. This will require at the street level a pressure of from 20 to 30 pounds in residence districts, and usually from 30 to 35 pounds in business districts, according to the character of the buildings.

For fire purposes the pressure required in the mains depends upon whether it is intended that fire streams shall be furnished directly from the hydrants or whether steam fire engines are to be used. In small cities and towns it is of the greatest advantage to supply fire streams without the use of engines, and in most such places this method is adopted, fire engines being sometimes kept in reserve, for extraordinary conflagrations. In pumping systems the most common arrangement is to maintain only a moderate pressure for ordinary service, and at times of fires to shut off the reservoir or standpipe if there be one, and to furnish the necessary fire pressure direct from the pumps.

In large cities hydrant fire pressure is not so common, but if the supply is by gravity, and has plenty of head, a hydrant fire pressure can profitably be furnished, at least for all except the densest portion of the city or for very large fires. If hydrant fire pressure is to be supplied it should not be less than 60 pounds for residence districts and 70 pounds for business districts. Pressures 20 pounds higher than these are to be desired. If dependence is to be placed on fire engines, as is usual in large cities, the domestic pressure of 25 to 30 pounds is sufficient.

The pressures here considered are the hydrant pressures at times of maximum consumption, and refer to any point in the distributing system. If such pressures are maintained at the most remote points and at the higher elevations, the pressures on the lower ground and at points nearer the pumps or reservoir will of course be considerably higher. In the case of gravity supplies much higher pressures may be possible, but on account of the increased cost of plumbing and piping to withstand high pressures it will not be desirable often to exceed 130 to 140 pounds.

90. Number and Size of Fire Streams. The number of fire streams which should be simultaneously available in any given town will obviously vary greatly with the character of the buildings, width of streets, etc. For average conditions the number may be calculated from the formula

$$y = 2.8\sqrt{x}, \qquad \textbf{(16)}$$

where y = number of streams, and x = population in thousands. About two-thirds of this number should be capable of being concentrated upon a single block or group of buildings.

In small cities and towns the requirements for fire protection may differ widely. For example, in a country town of 4,000 to 5,000 inhabitants, in which only a small mercantile business is carried on, the fire risk is not great, while in a town of the same size whose prosperity depends entirely upon two or three large factories, located, perhaps, in one large group of buildings, a fire would be a very serious matter. In the former case four or five fire streams would be sufficient, while in the latter case eight or ten should be supplied.

The number of fire streams is based upon a size of stream of about 250 gallons per minute, which is generally considered to be about right as an average value for good fire streams in business districts. For a residence district 175 to 200-gallon streams will usually meet the requirement. Fire hydrants must be sufficiently numerous and so located as to meet the requirements regarding number and size of fire streams set forth in the preceding paragraphs. Hydrants are one-way, two-way, three-way, etc., according to the number of hose connections provided. For most purposes the two-way hydrant is considered the most convenient, but in the dense portion of a large city, where many connections must be provided, three-way and four-way hydrants can be used to good advantage. Hydrants should, in any case, be numerous enough to enable the required number of streams to be furnished with a suitable nozzle pressure. At points where a large number of streams are required, fire cisterns are sometimes used instead of hydrants. These cisterns are fed by large pipes, and have an advantage over hydrants in that they allow several steamers to obtain their supply at one point.

For a 250-gallon stream the required nozzle pressure is 45 pounds and the loss of head per 100 feet of ordinary $2\frac{1}{2}$-inch hose is about 18 pounds (see Hydraulics), so that with a hydrant pressure of 100 pounds the length of hose to supply a 250-gallon stream cannot exceed 300 feet. A 175-gallon stream, with a 1-inch nozzle, requires 35 pounds nozzle pressure, and causes a loss of head of 9 pounds per 100 feet of hose. With a hydrant pressure of 100 pounds the length of hose in this case might be 700 feet. With a hydrant pressure of 75 pounds, which is quite common, a 250-gallon stream could not be supplied through a length of hose greater than about 200 feet, and a 175-gallon stream through a length greater than about 450 feet. Hence the general rule that hydrants should be so spaced that no line of hose should exceed 500 to 600 feet, and for at least half of the streams required at any point the length of hose should not exceed 250 to 350 feet, according to the hydrant pressure. These lengths cannot be much increased even where fire-engines are used. In outlying districts two two-way hydrants should be available at any point, with a distance of not more than 500 to 600 feet to the more remote of the two.

The most convenient location for hydrants is at the street intersections, as they are then readily accessible from four directions. In cities of moderate size the required number of streams can readily be supplied by locating a hydrant at each street intersection, but in large cities intermediate hydrants are often necessary. Thus if the blocks in Fig. 36 are 300 feet long in each direction, and a two-way hydrant is placed at each corner, then a fire at A could be served from eight hydrants, with a maximum length of hose of about 450 feet, giving sixteen good fire streams; while a fire at a street corner could be served from thirteen hydrants, eight of which would, however, require hose lengths of 600 feet. With blocks 600 feet by 300 feet, as in Fig. 37, a two-way hydrant at each intersection would supply not less than eight streams at any point, without exceeding 600 feet of hose. If only four streams are required, then one-fourth of the hydrants might be omitted, or every other hydrant in alternate streets, as hydrants 1, 2, and 3.

91. General Arrangement of the Pipe System. From the data on page 9 it is evident that the fire demand will largely govern in the design of the pipe system. This is more and more true the smaller the town or district considered, and for single blocks the ordinary consumption can practically be neglected. To supply long, narrow districts, the general scheme would be to furnish the water mainly through a single large pipe of gradually decreasing size, with small parallel and branch mains supplying the side streets. For broad areas, such as comprise the larger portions of most cities, the general arrangement usually adopted is to provide large mains at intervals of ¼ to ½ mile, and to fill in between these mains with smaller pipes, thus forming a gridiron system.

A general principle which should be kept in mind when laying out a system is to so arrange the large mains that the smaller cross mains may be fed from both ends, since a pipe so fed is equivalent to two pipes. It can furnish double the number of streams with the same loss of head, or the same number of streams with about one-fourth the loss of head, as when fed from one end only. This principle also makes it desirable to lay connecting pipes between separated districts, even when such pipes are not required for supplying local consumers. In the case of fire, each district may then be served from both ends. Dead ends are also objectionable on account of the stagnation which exists in the pipes and the deterioration of the water which is likely to ensue.

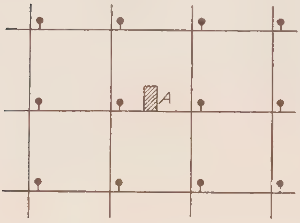
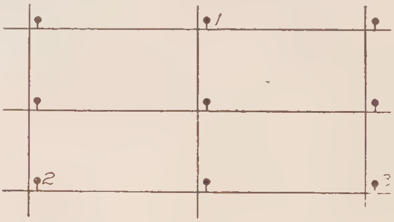


Fig. 36.



Location of Hydrants. Fig. 37.

The size of mains and cross lines in the gridiron system will depend largely upon the number of fire-streams required at any point. In small cities, and outlying districts of large cities, 6-inch cross mains with 8, 10, or 12-inch pipes at intervals of four to six blocks is a common arrangement. Four-inch pipe should never be used to supply hydrants except where the pipe is comparatively short and is fed from larger pipes at each end.

92. Calculation of the Pipe System. For the purpose of calculating the distributing system it is necessary to know the maximum rate of consumption for the entire city, and for large and small sections of the same, with suitable consideration for future growth.

The rate for the entire city will enable the main supply conduit, or the principal force main, to be determined. For calculating the main distributing pipes the city should be divided into relatively large districts, corresponding to the most probable location of such main arteries; then for the smaller pipes the demand for still smaller sections must be considered, and so on.

The maximum rate of consumption for the entire city has already been discussed in section 6. From the data there given the ordinary maximum rate is seen to be from 200 to 250 per cent of the yearly average. If the yearly average be 100 gallons per capita daily, the maximum ordinary rate will then be about 250 gallons per capita per day, or 0.17 gallon per capita per minute. The maximum fire rate, assuming 250-gallon streams, is $250 \times 2.8 \sqrt{x}, = 700 \sqrt{x}$ gallons per minute, where x = population in thousands. Thus for a population of 1,000 the ordinary maximum rate may be about 170 gallons per minute, while the fire rate is likely to be 700 gallons, or four times as much.

After estimating the maximum rate of consumption for the city as a whole, the same should be done for the several districts, the probable future population, the maximum ordinary rate, and the maximum fire demand being estimated for each district independently. The data so collected will enable the main distributing pipes to be calculated. The size of the cross mains and smaller pipes will be determined almost entirely by the local requirement as to fire streams. For all practical purposes an arrangement of 6-inch pipe in one direction, and 4-inch pipe crossing these, is ample for cities up to about 10,000 inhabitants, and six-inch pipes in both directions will suffice for populations up to about 50,000. For villages up to 1,000 or 2,000 population and the residence districts of small cities all but the general supply main may be 4-inch provided there are no dead ends and that there is a cross line at least every other block.

The size of the main supply pipe and the main branches feeding isolated districts can be calculated by the aid of Table No. 12 of Hydraulics giving the friction loss in pipes, an estimate of the maximum rate of demand having been made. For most cases the desirable velocities in the main pipes will be from 3 to 6 feet per second for the maximum rate of flow. The lower velocity is that suitable for a plant where the available head is limited and not much friction loss can be permitted, as for example in a gravity system where the elevation of the source is barely sufficient to furnish the desired pressure. The higher velocity is suitable where a considerable loss of head may be allowed, as for example in a gravity system with a high source, or in a pumping system where the fire pressure is furnished by pumps and only during the fire.

The number of fire streams of 250 gallons per minute each which can be supplied reasonably through pipes of different size are given in Table No. 13, the smaller number corresponding approximately to the lower velocity mentioned above and the larger the higher velocity. Where a pipe is fed from both directions double the number of streams can be supplied.

TABLE 13.

Number of Fire Streams Obtainable From Pipes of Various Sizes.

Size of pipe.	No. of 250-gal. streams.
4	1
6	1– 2
8	2– 4
10	3– 6
12	4– 8
16	8–16
20	12–24
24	18–36

Example. A town of 3,000 inhabitants is to be supplied through a force main 4,000 feet long. Assuming the average daily consumption to be 75 gallons per capita and that the town is of average character as regards fire demands, what would be a suitable size of main?

Referring to section 6, we find that the maximum rate for ordinary use may be taken at 180 per cent of the average, which would be $1.80 \times 75 = 135$ gallons. The rate per minute will be $\frac{135 \times 3,000}{24 \times 60} = 280$ gallons. The number of fire streams required is by formula 16 equal to $2.8 \sqrt{3} = 4.8$ or, say, 5. Each being assumed as 250 gallons the total rate will equal $280 + 5 \times 250 = 1,530$ gallons per minute, or practically equal to 6 fire streams. From the table No. 13 we see that a 10-inch pipe may be used if a considerable loss of head is permissible or a 12-inch pipe if but little loss is desired. From Table 12 of Hydraulics the actual loss of head in the 10-inch pipe for a flow of 1,530 gallons per minute is 16 feet per 1000, or 64 feet for the entire length of main. For a 12-inch pipe the loss is only about 6.5 feet per 1,000, or 26 feet total. Where the available head is not more than 150 feet the former loss would be too great.

93. Separate Services for Different Elevations. Where the different parts of a town vary considerably in elevation, it is frequently advisable to divide the distributing system into two or more independent portions, each serving an area or zone situated between certain limiting elevations. It often happens that only a small portion of a city is at a high elevation, and by thus separating the systems of distribution a comparatively small amount of water will need to be raised to the maximum height, the greater portion being pumped against a much lower pressure. By this arrangement a large saving can be effected in the expense of pumping, and the use of excessive pressures in the lower districts will also be avoided.

Various arrangements may be made for supplying the different zones. Each zone may be practically an independent system, with its own pumping station and perhaps its own source of supply; or the pumps of a higher zone may be supplied by a reservoir located at a high point in the next lower zone; or the pumps of the different zones may all be located at the same station and obtain their supply from the same source. In the gravity system a division is often made so that the lowest zone is supplied by gravity, while the upper zones are supplied by pumps.

94. Location of Pipes and Valves. The distributing pipes should be so located with respect to street lines as to be readily found and to avoid other structures as far as practicable. The center of the street being usually reserved for the sewer, the water pipes are placed at some fixed distance, usually from 5 to 10 feet from the center. The side chosen should be the same throughout. The north side of east and west streets will be warmer than the south side.

Valves should be introduced in the system at frequent intervals so that comparatively small sections can be shut off for purposes of repairs, connections, etc. As a general rule, wherever a small pipe branches from a large one, the former should be provided with a valve. Thus with 10 or 12-inch pipes feeding 6-inch pipes, each of the latter should have a stop valve at each end. At intersections of large pipes a valve in each branch is usually desirable. In a network of small pipes of uniform size, a valve in each line at each intersection, or four in all, is rather more than necessary, but two at each intersection, or a valve in each line every two blocks, answers very well.

Valves, like pipe lines, should be located systematically. They are usually located in range either with the property line or the curb line, but sometimes are placed in the cross walks.

95. Hydrants. The general location of hydrants has already been considered in section 90. In fixing upon the exact location, and the side of the street on which each should be placed, a detailed examination should be made and the location determined with reference to important buildings, convenience of access in case of fires, etc. Generally the hydrant is placed on the same side of the street as the pipe, and is connected to the larger of two pipes where there is a choice.

Hydrants are of two general types—the post hydrant, in which the barrel of the hydrant extends 2 or 3 feet above the ground surface, and the flush hydrant, in which the barrel and nozzle are covered by a cast-iron box flush with the surface. The former is more commonly used, and as it is much more readily found and more conveniently operated, it is to be preferred, except perhaps in the congested districts of large cities, or on narrow streets where all obstructions should be avoided. Post hydrants are set just back of the curb line; flush hydrants, either in the sidewalk or in the street.

Many styles of hydrants are on the market, most of which will give reasonably good service if properly handled. Reliability of operation is the first essential, but next in importance is the requirement that the frictional loss in the hydrant shall be small. All waterways should be ample, and sharp angles and sudden changes in size should be avoided as much as possible. Considerable difference exists in different hydrants in this respect, with a corresponding difference in the amount of pressure lost. In Fig. 38 are shown two forms of hydrants which illustrate the two general types of valves used—the gate valve and the compression valve. In ordering hydrants care should be taken to have the nozzles of the same standard as those used in adjoining large cities, so that connections can readily be made to fire apparatus which may be borrowed in emergencies.

When a hydrant is closed after use, the water remaining in the barrel must be drained out through a drip, so arranged as to open when the main valve is closed. This is an important feature of the design, as a hydrant is likely to freeze if not thoroughly drained. The escaping water may be led away through a small drain pipe to a sewer, or a considerable body of broken stone and gravel may be filled around the base, into which the water may be allowed to drain.

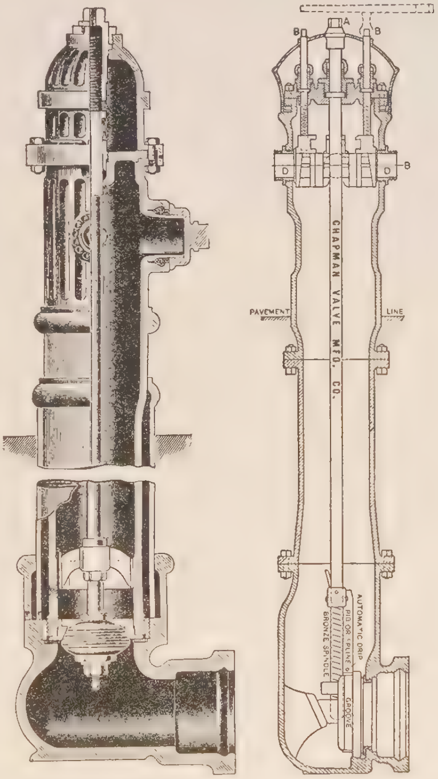


Fig. 38. Fire Hydrants.

In setting hydrants care should be taken to provide a firm base and to ram solidly back of the barrel. The hydrant branch should be covered at least as deep as the main, as this branch is essentially a dead end and is much more likely to freeze than the main itself.

96. Service Connections. Service pipes are usually from $\frac{3}{4}$ inch to 1 inch in diameter, and are made of lead, galvanized iron or tin-lined iron pipe. In making the connection between service pipe and main, the latter is tapped and a brass "corporation" cock screwed in. At the curb is usually placed another stop cock, with a suitable valve box, at which point the supply to the consumer is controlled.

Where pipes are laid in city streets, special care must be taken in backfilling and replacing the pavement. There is a wide difference of opinion as to the best method of backfilling, but probably the most certain way of getting the earth back without trouble from future settlement is by very thorough ramming of the material in a moist condition, but not wet. Such thorough ramming is difficult to secure, and it will usually be advisable to adopt the method of backfilling through a good depth of water. Hydrants are often deranged by being used for filling sprinkling carts. It is much preferable to provide water cranes for this purpose, numerous forms of which are on the market.

All constructive features pertaining to the distributing system should be carefully recorded on maps of adequate size and suitably indexed. The exact location of pipes, hydrants, and valves is of special importance. It will be convenient to have two sets of maps for this purpose—one on a small scale showing arrangement and size of piping and points of connection, and a set of large-scale maps, each one showing a comparatively small section of the system, on which the detailed information can be recorded.

OPERATION AND MAINTENANCE.

97. The maintenance of conduits and large pipe lines involves chiefly the work of cleaning and repairing. The various special structures should be frequently inspected to detect any sign of weakness, and in the case of large aqueducts the entire line should be regularly patrolled. If the water carries sediment and has a low velocity, the pipe line should be occasionally flushed by opening the blow-off valves.

Masonry conduits are likely to become coated with slime and organic growth, which will cause a large diminution of their carrying capacity, and if allowed to remain may affect the quality of the water. In such a case the aqueduct should be cleaned regularly once or twice a year, or at longer intervals, depending on the rapidity of the accumulations.

Large steel and cast-iron pipe lines will rarely need to be emptied for cleaning; but in some cases accumulations of organic growth have formed, which greatly obstructed the flow and which could not be removed by blowing off. In certain waters, particularly those relatively soft, the interior of cast-iron pipes corrode quite rapidly as explained elsewhere. This tuberculation, as it is called, often seriously reduces the carrying capacity of the pipe. The removal of such incrustation will restore a large part of the lost capacity, and may be a much more economical method of increasing the pressure in a system than by adding new pipes.

Large pipes can be cleaned by sending workmen through them, but ordinary pipes can only be cleaned by flushing or sending through them some form of mechanical scraper which nearly fills the pipe and which is propelled by the water pressure. Very badly corroded pipes have been successfully cleaned in this way.

To remove sediment from the pipe system use is made of blow-off valves or hydrants. Dead ends may need quite frequent flushing on account of odors and bad tastes developing in the stagnant water. Large leaks in mains will quickly make themselves known, especially if a recording pressure gauge is in use. Prompt action in shutting off the supply is often necessary to prevent heavy damage. Small leaks, if occurring in clay soil, will usually be indicated by the appearance of water at the surface, but in porous soils, and especially near sewers or drains, quite large leaks may go unnoticed for years.

A serious form of corrosion which has given trouble in many cities is the electrolysis which is caused by return currents from single-trolley electric railways. In this system the return current is supposed to pass through the rails, but as these are not insulated, a portion passes through the earth to neighboring pipes or other conductors leading in the right direction. This current then flows along the pipe with more or less resistance until it reaches a neighborhood where the rails or some other conductors are of lower potential than the pipe, this being usually in the vicinity of the power station. The current then leaves the pipe, and in so doing sets up corrosive electrolytic action.

Electrolytic corrosion is in some cases so rapid that pipes are practically eaten through in three to four years, and some of the worst cases have occurred where the pressure is but $1\frac{1}{2}$ volts. The remedies for electrolysis should apparently rest entirely with the railway companies. A very important aid in preventing electrolysis is the construction of a good return conductor by means of good rail bonding and the use of adequate return wires. Then in those districts where the pipes are of higher potential than the rails, if good, low resistance connections are made between rails and pipes, or from pipes to special return wires, the current will leave the pipes without passing into the ground and without causing trouble. Voltmeter tests between pipes and rails, at various points over the city, will determine the danger area.

Not infrequently considerable trouble arises from the freezing of service pipes which are not placed at a sufficient depth. Occasionally, also, small mains are frozen. Where the proper facilities exist the best way to thaw frozen pipes is by warming them with an electric current. For thawing service pipes a current of 200 to 300 amperes at a pressure of 50 volts is satisfactory, and will ordinarily thaw a pipe in from 20 to 30 minutes. The current can conveniently be taken from electric-light wires and reduced by a transformer.

Where the electrical method cannot be used steam may be employed, not only to warm the pipe, but to excavate through the frozen ground in a way similar to the operation of the water jet. The pipe may thus be reached at points 4 to 5 feet apart and gradually thawed out. Service pipes are often thawed by the use of a small steam pipe inserted in the service pipe through the house end, or from an opening at an excavation outside. Ground may be thawed by maintaining a fire on the surface for several hours, or more readily by the use of a gas flame projected against the soil.

Valves should be inspected occasionally to detect leakage and to ascertain if they are in working order and the boxes clean. Fire hydrants require very careful attention, especially in cold climates, as it is of the greatest importance that they be at all times available. The chief trouble with fire hydrants is from the freezing of the valves due to imperfect drainage, although a hydrant branch sometimes freezes up.

Hydrants should be carefully examined on the approach of cold weather and put in good condition. Valves should be tight and the hydrant thoroughly drained. If so located that the hydrant cannot be drained, it should be pumped out each time after being used. To ascertain if a hydrant is drained, a lead weight tied to a graduated cord can be let down through a nozzle. Hydrants should never be opened unnecessarily in cold weather, and never by others than those responsible for their condition. In very cold climates it is found desirable after using a hydrant to oil the packing and the nut at the top with kerosene in order to prevent sticking of the valve and nut.

To thaw frozen hydrants, a small portable steam boiler is commonly employed, which is provided with a length of hose for conducting steam to the bottom of the hydrant. Hot water may also be used, and for mild cases a little salt may be effective. After thawing, the water should always be pumped out.

In the management of the pumping station the best results can only be obtained by employing thoroughly competent men. The item most susceptible of variation is the cost of coal, and every effort should be made to reduce this to the lowest practicable limit. A daily record should be kept of the weight of coal and of ashes, so that the efficiency of the service can be known at all times. Reserve machinery should be operated frequently to make sure it is in good condition and can be started when called for. This is especially important where it is depended upon for fire pressure.

Records should, of course, be kept of the amount of water pumped per day, and the pressure maintained; also of the time during which special fire pressure is furnished, and the amount of water pumped at this pressure. Recording pressure gauges are of the greatest value in maintaining the efficiency of a plant.

The maintenance of earthen reservoirs calls for little more than has already been mentioned in section 76. The cleaning of such reservoirs may need to be done frequently. It is usually accomplished by flushing out the mud through the waste pipe by means of a hose, as in the cleaning of settling basins. Standpipes and tanks may require occasional flushing or blowing out, and will need to be repainted at intervals of a few years. They should also be inspected for signs of excessive corrosion or of electrolysis, and for any indication of weakness or wear at the base. Wooden tanks need rigid and frequent inspection to ascertain the condition of the wood and of the hoops. One or two of the latter will probably need to be occasionally removed to determine this point.

98. Detection and Prevention of Waste. From the data given in section 6 it was made evident that a very large percentage of the water supplied to American cities is wasted by the consumer and lost by leakage. In many cities the consumption of water is easily double the amount which can possibly be made use of, and in a very large proportion of them the wastage is fully one-third of the entire quantity supplied. This excessive use of water not only increases the cost of pumping unnecessarily, but adds to the expense in all parts of a waterworks system.

Unquestionably the easiest and most rational method of preventing the waste of water is by the use of meters, so that each consumer will pay for what he uses. It furnishes also the most equitable basis for charging up the cost of service, as by any other system the careful user is forced to pay for the water wasted by his careless neighbor. The use of meters is becoming much more general, and in most cities the larger consumers, at least, are now metered; but a very large part of the loss or waste is due to the small consumer, so that the full benefit of the system will not be felt until the use of meters becomes general. Usually much opposition is raised to the introduction of meters, but after they have been put into use the results are commonly such as to cause them to be greatly favored by the community. As a system of waste prevention it is always in service, and for that reason is far superior to any system of inspection. In nearly all cases the decrease in cost of supplying water after the adoption of meters much more than balances the cost of the meters.

If meters are not used, some method of inspection is highly desirable whereby the most serious cases of waste can be detected and the consumption kept within reasonable limits. The most common method is a house-to-house inspection, carried out one or more times per year for the purpose of examining the plumbing fixtures. Any leaky or imperfect fixture is ordered repaired, and the premises re-inspected shortly to make sure that the order has been complied with. Persistent refusal is followed by the shutting off of the supply.

One of the weak points of the meter system is that it fails to detect leaks in the mains or in the services beyond the meters. To localize a leak in a main, a waterphone may be used, which consists of a staff of wood or iron having at one end a diaphragm and ear piece similar to a telephone receiver. The staff is placed against the pavement over the pipe at various points, and the ear applied to the receiver, when any sound made by a leak is readily perceived.

Many different kinds of meters are on the market, most of which will give satisfactory service if properly treated, and many of them have been thoroughly tested by years of use. No new form of meter should be adopted without thorough and long continued tests, and in all cases it is well to specify the desired requirements of a meter, and to test all new meters, in order to insure uniformly good workmanship.

The general requirements of a meter are—a fair degree of accuracy, ability to register approximately quite small rates of flow, suitable capacity for a given loss of head, durability, and low cost. All of these requirements except that of durability can readily be determined by a brief test. Some notion of the durability can also be had by a careful inspection of the parts, and by running a meter at a rapid rate for a considerable period and again determining its accuracy and sensitiveness. Maintained accuracy, accessibility, and ease of repairs are the most important qualities of a meter.

Meters should be so designed that the various parts will be easily accessible and readily replaced, and the moving parts protected from serious injury by frost. The latter object is usually accomplished by frost bottoms of cast iron, or cast-iron cases, made so as to be more easily broken than other and more costly parts of the meter.

99. Water Rates. The several services performed by a waterworks are: (1) to furnish water for private use; (2) to furnish water for public use on the streets, and for sewers, fountains, public buildings, etc.; and (3) to furnish fire protection to property. In (1) and (2) the cost of service may be considered approximately proportional to the quantity of water supplied, but in (3) it is out of all proportion to the amount of water used, for while the cost of construction is greatly affected, the total amount of water consumed is slight. The extra cost involved in furnishing adequate fire protection is due to largely increased pumping capacity, increased size of mains, reservoirs, or standpipes, cost of hydrants, and increased cost of maintenance. Estimates of careful observers place the proportion of cost chargeable to fire protection at one-third or one-half the entire cost.

The sources of revenue are the water rates and the fund received by general taxation. The former are paid by those who use the water, and more or less in proportion to the amount used. The latter are paid by assessment on all taxable property. If the revenue be so raised that each interest served be charged according to the cost of the service, it would appear from the preceding section that the cost of furnishing water to private consumers should be paid by water rates; that the cost of supplying water for public purposes should be paid by taxation and according to the amount of water used; and that the cost of fire protection should also be met by taxation, since the individual is benefited by reason of the protection afforded to property.

The exact proportion of the revenue which should be derived from each source depends much upon local conditions, such as size of town, character of supply, etc. In many small towns the works are primarily installed for fire-protection purposes, in which case nearly all the expense should be met by taxation. It is also good policy to begin with fairly low water rates, so as to encourage the use of water, but to enable this to be done a large proportion of the expense will have to be met for a few years by taxation.

The proportion of the revenue to be derived from private consumers requires careful consideration in its adjustment. The most equitable method of apportioning the cost is by the meter system. In fixing rates under this system, allowance should be made for the fact that quite a large percentage of the water recorded at the pumping station cannot be accounted for, and rates per unit of volumes registered by the meters must be correspondingly raised.

Meter rates are usually graduated, that is, a less rate is charged for large quantities than for small ones. This is partly on the ground that the cost of meter maintenance, keeping of accounts, etc., is proportionally greater for small quantities, and partly by reason of the policy of encouraging the operation of factories which contribute largely to the general prosperity of the community, and which may require large amounts of water. In establishing a graduated schedule, it should be so made that the lower rate shall apply only to the additional water used beyond the limit of the next higher rate. A good example of such a schedule is as follows:

For the first	5,000	cu. ft.	per	6 months,	20	cts.	per	100	cu. ft.
“ “ next	15,000	“	“	“	10	“	“	100	“
“ “ “	10,000	“	“	“	5	“	“	100	“
“ “ “	30,000	“	“	“	3	“	“	100	“
“ “ “	30,000	“	“	“	2	“	“	100	“
“ over	90,000	“	“	“	5	“	“	100	“

A minimum charge of $2.00 per 6 months is made.

An objection to the meter system which is often advanced is that it discourages the use of sufficient water for sanitary purposes, but this is entirely obviated by making a small minimum charge, such as given above, which will be enough to allow the use of an abundance of water for sanitary purposes, and at the same time will cover the expense of meter maintenance.

Most cities meter the larger consumers, but comparatively few have yet introduced the full meter system. In such cases private houses are charged mainly by the fixture. Usually a minimum family rate is charged for kitchen use, then an additional rate for each bath tub, water closet, wash bowl, stable hose, lawn hose, etc., with often other variations depending upon the number of rooms, number of occupants of the house, etc. Little data exists as to the actual amount of water used by different fixtures, and the rates are largely arbitrary.

PURIFICATION OF WATER.

100. Object and Methods. In the purification of public water supplies the primary object is usually to remove from the water any traces of pollution that may give rise to disease, or, in general, to remove any disease germs that may possibly infect the supply. It is often important also to remove the suspended matter where the water is turbid. Sometimes also the water contains so much dissolved mineral matter that it is desirable to remove a part of this to render the water more suitable for manufacturing as well as for domestic purposes. Thus, a very hard water is undesirable to use for boiler purposes as well as for culinary and laundry uses.

The various processes of purification may be divided into two general groups: (1) Those for the removal of suspended impurities, and (2) Those for the removal of dissolved impurities. Of the first class there are two general processes, sedimentation and filtration, both of which may be called natural processes. By sedimentation, water may be more or less freed of its suspended matters, including the bacteria, the efficiency of the treatment depending much upon the element of time. The process is carried out artificially in large storage reservoirs or in small special settling basins.

Filtration is accomplished in different ways. The most common is by means of the artificial sand filter bed, either as contained in masonry basins of large size, or confined in small tanks as in the so-called mechanical filters. The chief object is in all cases the removal of the suspended matters, and in most public supplies particular attention is paid to the removal of bacteria. The processes for the removal of dissolved impurities include the softening process, in which lime and magnesia are removed by chemical precipitation, and the process for the removal of iron in a similar manner. Such methods usually involve subsequent sedimentation or filtration for the removal of the precipitate.

SEDIMENTATION.

101. In streams such as would be considered suitable as sources of supply the sediment is principally of an inorganic nature, consisting of particles of sand and clay of various sizes. The amount and character of the sediment varies greatly from time to time; it depends largely upon the stage of water in the different tributaries, and upon the geological character of the various parts of the drainage area. In the Ohio River water at Louisville it varies from 1 to 5,000 parts per million, ranging ordinarily from 100 to 1,000; the bacteria varies from a few hundred per cubic centimeter to as high as 50,000. The size of the suspended particles also varies greatly. In some waters the finer particles of clay are less than 0.00001 inch in diameter, which is smaller even than bacteria. This great variation in amount and kind of sediment constitutes one of the most troublesome factors in connection with purification works for river supplies.

Where the body of quiescent water is sufficiently large, and the period of repose sufficiently long, this action of sedimentation becomes practically perfect, and a clear and greatly improved water is the result. Artificially, such high efficiency is often obtained where the water is collected in large impounding reservoirs holding several months' supply. Where, however, the supply is taken directly from a large sediment-bearing stream, very large reservoirs are usually impracticable on account of the great cost; and the period of time during which sedimentation can be operative must therefore be limited to a few days or even to a few hours. Such a limited amount of sedimentation is, however, of much value.

Where a water contains little that is objectionable besides the inorganic sediment, a degree of purification can often be obtained by mere sedimentation which will render the water fairly acceptable. In many instances, however, a satisfactory water cannot be obtained without subsequent filtration; but in this case the process of sedimentation constitutes a very valuable and almost indispensable prerequisite to the final treatment. For a sewage-polluted water, sedimentation alone is an inadequate treatment, as the bacteria are not eliminated in sufficient numbers to insure safety.

Sedimentation may be employed as a preliminary treatment of a water that is to be filtered or it may be used as the final and only treatment. In the former case a fully clarified water is not essential, but in the latter case it is greatly to be desired, although not always possible.

There are two methods to be considered: (1) Plain sedimentation; (2) Sedimentation with the addition of a coagulant. For plain sedimentation a period of 24 hours' subsidence is about the minimum limit adopted, but this will seldom give a clear water. A considerably longer time is often necessary to give acceptable results. For some waters it requires weeks and even months to remove all the turbidity, while for others a settlement of a day or two accomplishes fairly good results. If the amount of suspended matter is measured by weight, a large proportion will settle in one or two days; but the reduction in turbidity is not correspondingly great, as it is the finer portions which exert the greatest influence upon the appearance of a water. In the case of the Mississippi at St. Louis, it is practically impossible to clarify the water in the spring by simple sedimentation, owing to the attenuated condition of the clay particles.

There is a marked degree of bacterial purification in sedimentation, yet it should be kept in mind that such a method is extremely hazardous, especially where the water supply is subject to any sewage pollution. Experience shows that sedimentation alone is insufficient to protect a city against a polluted water supply.

Various chemicals when added to water will combine with certain substances ordinarily present, forming precipitates which are more or less gelatinous in character. These act as coagulants to collect the finely divided suspended matter into relatively large masses which are much more readily removed by sedimentation or filtration. Color may also frequently be removed to a large extent by this treatment. If a water can be satisfactorily purified the greater part of the year by plain sedimentation, the use of a coagulant at other times as an aid in the process is well worth consideration.

Several substances can be used as coagulants. That most commonly employed is sulphate of alumina. When this substance is introduced into water containing carbonates and bicarbonates of lime and magnesia, it is decomposed, the sulphuric acid forming sulphates with the lime and magnesia, while the carbonic acid is set free, and the alumina unites with water to form a bulky gelatinous hydrate which constitutes the coagulating agent. If the water does not naturally contain a sufficient amount of alkalinity to decompose the necessary amount of coagulant, lime should be previously added to the water. The regulation of this matter must be put into the hands of an expert, as an excess of alum in the water is very undesirable if not actually dangerous.

The amount of chemical required depends upon the amount and character of the sediment, upon the degree of purification desired, and upon the time of settlement. It varies in practice from about $\frac{3}{4}$ grain to 3 or 4 grains per gallon. The proper amount can only be determined by experiment.

The rate of sedimentation depends greatly upon the amount of coagulant employed. It takes place much more quickly than where no coagulant is used, so that a large part of the action will occur in a few hours. With a fair amount of coagulant, the sediment remaining after 24 hours' subsidence will settle very slowly; and this period may be taken as about the maximum economical figure. Much less time than this can be used in many cases. Where the water contains large amounts of sediment, it will often be more economical to allow the coarser particles to settle before applying the coagulant. This will reduce the cost of chemicals and give a more satisfactory result.

Settling basins are constructed in accordance with the same general principles as other reservoirs; in fact, in many cases distributing reservoirs or storage reservoirs act also as settling basins Where, however, but a short time is allowed for settling, and reservoirs are intended for that special purpose, there are differences in detail which should be considered. Settling basins are usually supplied with water by means of low-service pumps, and from the basins the water flows into a clear-water reservoir, or to a pump well, or to filters, as the case may be.

102. Methods of Operation. There are two general methods of operating settling basins: (1) the constant-flow method, and (2) the intermittent or fill-and-draw method. In the former the water is allowed to flow at a very slow velocity through one or more reservoirs, during which time the settling takes place. In the latter the water is let into a basin and allowed to remain quiescent during the period of subsidence. It is then drawn off to as low a level as efficient clarification has taken place, and the basin refilled. It is probable that certain waters can be treated best by one system and others by the other system, but this is a matter which can only be determined by experiment. The method of fill-and-draw, used at St. Louis, is thought by the engineers in charge to be more suitable for conditions at that place.

In several plants using Missouri River water the constant-flow system is used. At Cincinnati, Ohio, the fill-and-draw method is used, but it is stated that this is on account of matters pertaining to the form of the basins which are purely local in character. In the fill-and-draw method no settlement of fine particles can commence until the operation of filling is completed, which condition materially reduces the time of subsidence. On the other hand, the water becomes more quiet than in the other process, and this operates to its advantage. If the basins are operated on the constant system, a single basin can be made to suffice—an arrangement quite suitable for a relatively clear water where sedimentation is a secondary matter, or merely a preparation for filtration. If there is much sediment, at least two basins are needed, in order that one may be cleaned without interrupting the supply. In case a coagulant is used after partial sedimentation, two basins would be necessary and three would be desirable. With the fill-and-draw method, the number becomes a question of economical construction and operation. This will usually be from 4 to 6.

For a single rectangular basin of given area the square is the most economical form. For a number of basins they should be made rectangular in shape with a width about three-fifths of the length and arranged side by side in one row.

In general, settling basins are built similar to ordinary reservoirs, partly in excavation and partly by embankment, so as to secure the greatest economy. Earthen slopes will usually be cheaper than masonry walls, but with the fill-and-draw method the former have the disadvantage of exposing the mud at each period of emptying. They are, however, more often used in spite of this. The depth of basins is made about such as to give the most economical construction, very shallow basins being avoided. The time of settlement is found not to be materially affected by depth.

Fig. 39 illustrates the arrangement of the large St. Louis basins. The basins are of 22,000,000 gallons drawing capacity each. They are built with masonry side and partition walls, and linings of concrete, on about 18 inches of puddle. Through the center runs a ditch having a slope of 1 per cent, and leading to a 24-inch drain pipe at the east end. The floor also slopes towards this ditch from both sides. The filling is done through a 60-inch pipe leading from a filling conduit of masonry. The drawing is done at the east end through two sluiceways, 4 by 5 feet in size, which take the water from about 5 feet above the bottom. The water passes thence through a 60-inch pipe into the main delivery conduit.

In the continuous flow system the object to be obtained is the distribution of the water on entering as evenly as may be across one side or one end of the basin so that it shall enter with as little disturbance as possible; then to draw it off in a similar manner from the opposite side, and from the stratum of clearest water. As far as possible all parts of the water should remain in the basin equal lengths of time, and all strong currents should be avoided. The ordinary inlet is usually a single large pipe laid through the embankment, or a single sluice gate in a gate chamber built in the walls. A better distribution of the water could be obtained by means of several inlets, or several branches from a single inlet pipe. The withdrawal of water in this system should take place from near the surface. Broad weirs formed in the wall, or made of iron troughs, are frequently used.

In the fill-and-draw system the inlet is arranged in the simplest way, as in an ordinary reservoir. The position of the outlet is of more importance. If but a single one is used, it will need to be at the lowest point of outflow, and so will not draw from the clearest stratum except near the end of the operation. The difference in clearness at different depths after 24 hours' subsidence or more is, however, not very great. To enable the sediment to be removed, the bottom of the basin should be made slightly sloping (1 to 2 per cent grade) towards a central drain leading to an outlet gate or to a drain pipe. The mud is removed by flushing into it the drain by

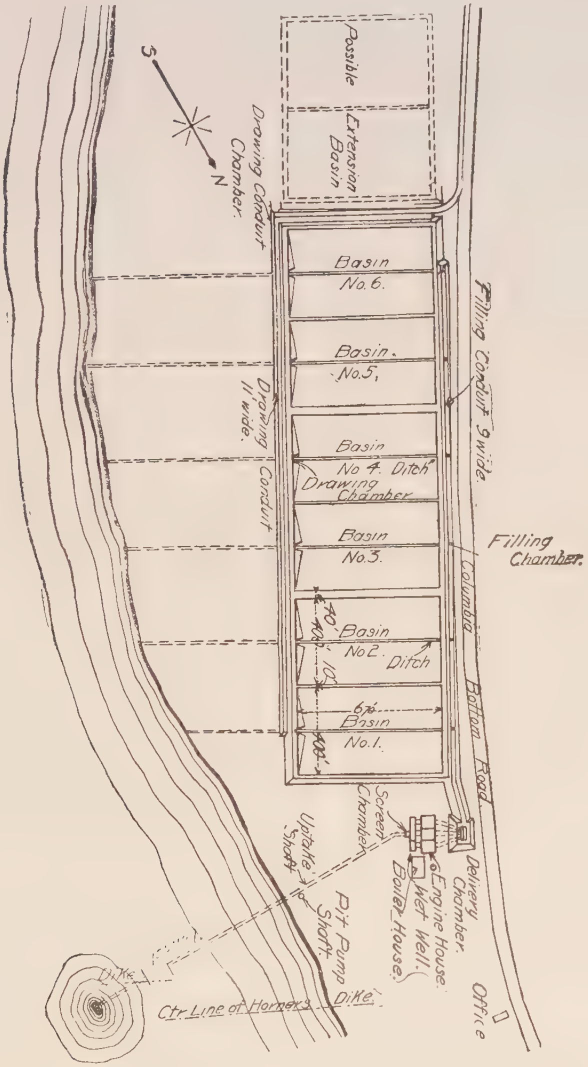


Fig. 39. St. Louis Settling Basin.

means of a hose stream, supplied from a high-pressure main. The cleaning is done at intervals depending entirely upon the local conditions, and may be every month or so, or only at intervals of years. The longer the mud is allowed to remain the more compact it becomes and the more difficult to remove, but the change in compactness takes place quite slowly.

Where the basins are operated on the continuous-flow system and the water passes from them directly to the pumps, it is necessary to construct a small clear-water or pump well to avoid the necessity of too frequent adjustment of the rate of supply to the basins.

SLOW SAND FILTRATION.

103. The first filter of which we have any record was established in 1829 for the Chelsea Water Company, of London. The chief object of this filter was to remove turbidity, and in this it was a success. Its value in improving the water from a hygienic standpoint was also appreciated, although the principles underlying its action were not understood until some years later. As a consequence of the good results obtained from this filter, the filtration of all river-water supplies of London was made compulsory in 1855. Within the last fifteen or twenty years the use of sand filters has become almost universal abroad wherever surface waters are used. In Germany it is made compulsory. In the United States it is only very recently that this subject has received the attention that it merits, but within the last few years many small cities and several large ones have installed efficient filter plants.

In the slow sand filter the sand bed is constructed in large watertight reservoirs, either open or covered, each having usually an area of from $\frac{1}{2}$ to $1\frac{1}{2}$ acres. On the bottom of the reservoir is first laid a system of drains, then above this are placed successive layers of broken stone and gravel of decreasing size, and finally a bed of from 2 to 5 feet of sand which forms the true filter. The water flows by gravity, or is pumped, upon the filter, passes through the under drains to a collecting well, and thence to the consumer. Water containing much sediment is usually first passed through settling basins, where a large part of the sediment is removed.

As the water filters through the sand, the friction causes some loss of head, which gradually increases as the filter becomes clogged with foreign matter. The rate of filtration is, however, maintained nearly uniform by suitable regulating devices which vary the head according to the resistance. When the working head has reached a certain fixed limit of a few feet, the water is shut off, the filter drained, and the surface cleaned by removing a thin layer of clogged sand. The operation is then resumed. Before the thickness of the sand layer becomes too greatly reduced, clean sand is added sufficient to restore the filter to its original depth.

Besides the method of construction, the chief characteristic of this system of filtration is the slow rate of operation, usually not exceeding 2 or 3 million gallons per acre per day.

The chief features to consider in this form of filter are the proper construction of sand bed and drains, the rate of filtration and its regulation, the loss of head, cleaning of beds, washing of sand, and the control of the operation by bacteriological tests.

104. Rate of Filtration. In the design of a filter plant the first question to be settled is the rate of filtration which shall be adopted. The higher the rate the less the area required and hence the less will be the first cost; but the cost of operation is not greatly affected by the rate. In general high rates of filtration will give less efficiency than low rates, but until the rate exceeds a certain amount the difference in efficiency is small.

Rates of filtration are in this country usually stated in terms of gallons per acre per day or per hour. The experience of European works has resulted in the adoption of a rate, for most places, of between 2 and 3 million gallons per acre per day, but in this country somewhat higher rates have been favored. Probably 3 or 4 million gallons is as high as it would in general be advisable to go in the design of a new plant. If subsequent operation shows that a higher rate can be adopted with efficiency and economy, the fact can be taken advantage of as the demand for water increases. It should not be overlooked that there may be cases, where, with a moderately polluted water, all the practical benefits of filtration can be secured at rates much higher than are usually employed. Each case demands independent consideration in order that the best and most economical solution may be arrived at. Sudden changes of rate are apt to produce disturbances in the filter and to give a reduced efficiency. In practice, absolute uniformity of operation is unnecessary, but sudden changes in rate should be avoided, and especially any large increase above the normal.

105. Capacity. The standard rate having been determined, the required net working capacity will be equal to the maximum rate of delivery divided by the assumed rate of filtration. To economize area and to avoid rapid changes in rate, a clear-water reservoir should be provided. The best size for this will depend on local conditions, but it will usually be desirable to have it of sufficient capacity to equalize the demand throughout the day. It will then be necessary to vary the rate of filtration only to accord with the daily variations in consumption. In section 6 it was shown that the maximum daily rate of consumption is likely to be about 150 per cent of the average, and with a clear-water reservoir of the capacity mentioned above, the filters must be designed to deliver at this maximum daily rate.

In addition to the area as above found, a reserve area for cleaning must be provided. For small works this will be one bed; for works containing several beds it will be necessary to allow one bed for each 5 to 10 beds, depending on the frequency of scraping and the time required for putting a filter into operation after cleaning. The proper size of beds is chiefly a question of economical construction. The larger the beds the less the cost per acre, but the greater will be the area out of service in the one or more reserve beds. Ordinarily the size for a considerable number of beds is from 1 to 1.5 acres for open beds, and from .4 to .8 acres for covered beds. For small total areas of .5 to 1 acre three beds would ordinarily be used, and for still smaller areas two beds. The economical number can in any case be determined by comparative estimates, but under ordinary conditions the number should be about as follows:

For a total area of 1 acre 3 to 4 beds.
" " " 3 " 5 " 6 "
" " " 6 " 7 " 10 "

Filter beds are usually made rectangular in form and arranged side by side in one or two rows according to the number. In general construction a filter basin is built in a way similar to small distributing reservoirs. (See section 76.) Earth embankments for the sides are cheaper than masonry walls, but require more ground. If the filters are covered, masonry walls are usually employed. Particular care must be taken to render the basin water-tight both on the bottom and at the sides. Cracks in division walls are likely to admit unfiltered water to the under drains and should be especially guarded against.

The depth of open filters is made only sufficient to contain the necessary depth of filtering materials and water, as explained subsequently, and still have a margin of 2 or 3 feet from the water surface to top of the embankment. This will give a total depth of 9 or 10 feet. In closed filters the distance from top of sand to cover must be sufficient to give head room for workmen when cleaning the filter, a distance of about 6 feet.

Covers for filters are constructed of the same general form and arrangement as described in section 78. Masonry or concrete vaulting is usually employed, although wood has been used; but the latter does not afford as good a protection from freezing or from summer heat. Admission for workmen is provided by a gangway leading from an opening at a point where the vaulting is raised. Walls and piers should be built with small offsets near the bottom in order to insure good filtration at that point. A covered filter is illustrated in Fig. 33.

The principal reason for covering filters is to avoid the difficulties connected with the operation of open filters in winter. To clean filters when covered with ice is a troublesome and expensive operation, requiring the removal of the ice or the use of special methods giving inferior results. If the filters are drained for cleaning, trouble also arises from the freezing of the sand. The cleaning of beds is thus not likely to be done as promptly as desirable, and the result of winter operation will be a decreased effective area and a lowered efficiency. Whether covers should be used depends upon the extent to which ice will form, the frequency of the occurrence of thaws which will enable a filter to be properly cleaned, and the length of time between cleanings as determined by the character of the water.

106. Filter Sand. Experiments show that very fine sand is considerably more efficient in removing bacteria than ordinary or coarse sand, but within the ordinary limits of size there is but little difference in efficiency. The finer sands, however, cause a steadier action and prevent disturbances due to scraping; they also cause a greater loss of head in the filter, and so make the action more uniform over the filter area. On the other hand, fine sand becomes clogged sooner than coarse and involves therefore more expense in cleaning.

It is desirable that a sand be fairly uniform in grain. If the particles vary greatly in size, it will be difficult to wash, and in fact will have much of the finer particles removed in the process, thus increasing the effective size. It is especially important that the sand should be of the same grade in all parts of the same filter in order that the frictional resistance, and therefore the rate of filtration, shall be uniform. In designing a filter it should be noted that the sand forms the filtering medium; the gravel serves simply to collect the filtered water with little resistance to flow. There is no object in having the main body of sand of different sizes.

The depth of sand must be sufficient to form an effective filter and, besides, to allow of several scrapings without renewing the sand. The effect of deep beds is similar to that of fine sand in steadying the action of a filter, and it has been clearly shown that the operation of beds 4 to 5 feet thick is not so much affected as that of beds 1 to 2 feet thick by such disturbances as variations in rate, scraping of beds, etc. A depth of 3 feet is about right, with one foot allowed for scraping.

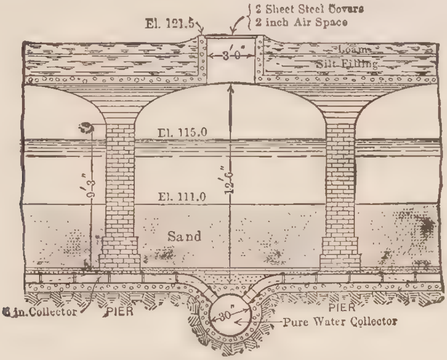
The depth of water on the filter should be sufficient to enable the desired maximum head to be used without reducing the pressure in the filter below atmospheric; and as the resistance is nearly all at the surface of the sand, the depth must be about equal to the maximum head to be used in forcing the water through the filter. The depth must also be greater than the thickest ice likely to form. Beyond these limiting depths any increase serves only to increase the expense of construction.

107. Drainage Systems. To collect the filtered water a system of under drains is necessary. The important points to be considered in its design are durability and freedom from derangement, and that the loss of head therein shall be small. The system of drains usually consists of a large central drain running the length of the filter, and branch drains at right angles thereto placed at regular intervals, usually of 8 to 12 feet. The central drain may be either of large vitrified pipe, as in Fig. 40, or of masonry. The branch drains are usually of 4 to 8-inch round or special tile, laid with open joints.

To conduct the water to the lateral drains, coarse gravel an inch or two in diameter is filled about the drains and spread in a layer of

6 inches or more in depth evenly over the floor of the filter, or, if the bottom of the filter is irregular, it may be arranged as shown in Fig. 40. Above this coarse gravel are then placed three or four layers of finer gravel, each successive layer being finer in size, but not so fine as to settle into the previously laid layer. The last layer is made fine enough to support the sand. The thickness of these layers need be only 2 or 3 inches if carefully laid, or just sufficient to insure that the next layer below is well covered.

The gravel used should be carefully screened and, if dirty, washed. It is readily sized by revolving or fixed screens, using for this purpose three or four different sizes. The smallest should have about a $\frac{3}{16}$-inch mesh, and each larger size about double the size of mesh of the next preceding. As a filter becomes clogged the head necessary to cause filtration at the assumed rate increases. By allowing the head to increase to a high figure the filter can be operated longer without scraping and so a saving in operation effected. On the other hand, high losses of head require more pumping, a greater depth of filter, and have a detrimental effect in compacting the sand. A maximum loss of head of 4 or 5 feet may be taken as good practice.



Section through Main Drain.

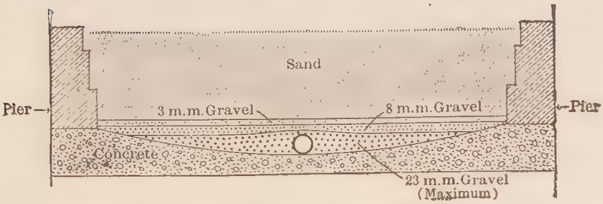


Fig. 40. Details of Drains, Albany Filter Beds.

108. Arrangement of Inlet and Outlet. Water is admitted to the filter through a single branch main at about the level of the surface of the sand. The flow is usually controlled by a valve operated by a float, so as to maintain the water in the filter at a constant level. A gate valve is provided in addition, to enable the water to be completely shut off at any time. If the water level on the filter is kept constant, the rate of filtration must be regulated, as the filter becomes clogged, by lowering the water level or reducing the pressure at the outlet. In the older filters no arrangement was provided for regulating each filter independently, but each was connected to the clear-water well by a short pipe fitted with an ordinary valve. The head on all filters was consequently always the same, except as it might be controlled by throttling at the valves. The effect of unequal heads on the rate of filtration, where some of the filters might be freshly cleaned and others badly clogged, can readily be imagined.

The regulation of head requires, first, some form of measuring device, such as a weir or orifice, by which the rate of filtration can be ascertained at any time by floats and indicators; and, second, the controlling of the head on this weir or orifice either by hand or automatically. Floats are also required to show the level on the filter and the head in the main drain, the difference of which is the working head on the filter. The apparatus for regulation is placed in one or more chambers with which the main drain of the filter connects.

Automatic regulators for delivering water at a constant rate are in use in a number of places. They usually consist of a weir in the form of a telescopic tube which is supported by means of a float in the chamber connecting with the under drain. By adjusting the float, the edge of the weir can be maintained at any desired distance below the water surface. Besides the inlet and outlet pipes, a drain pipe must be provided through which the water may be drawn off. This is usually connected with the chamber into which the main drain opens. An overflow pipe is also necessary to provide against any failure on the part of the inlet regulator. This connects with the drain pipe.

Arrangements should be made for wasting the filtered water in case it should be necessary, also for drawing off the water from above a filter down close to the sand layer in order to save time in emptying; and facilities should be provided for sampling water from various points in the system. By-passes should be provided to enable either settling basin or filters to be cut out if necessity arises. For furnishing water for sand-washing and various purposes, connection must be made with high-pressure mains.

109. Cleaning Filters. When a filter has become clogged and has reached its highest allowable loss of head, it is drained and then cleaned by removing by means of broad thin shovels a layer of clogged sand from ½ to 1 inch in thickness. The surface is then smoothed with a rake. The sand is removed from the filter by means of wheelbarrows or small cranes, and deposited at a convenient point where it is cleaned after a considerable quantity has accumulated—or is wasted in case it is cheaper to use new sand than to clean the old. After scraping, the filter is filled, preferably from below, with filtered water until covered 2 or 3 inches deep; then raw water is run on to the usual depth, and the filter again started into action. At intervals of a year or so, and before the layer of sand has been reduced below a desirable minimum thickness, the bed is restored to its original depth by the addition of clean sand. After cleaning and filling, the filter should be started slowly and gradually. The sand that has been removed is allowed to accumulate until it is desired to replace it in the bed. Before replacing it, however, it is washed to free it of the accumulated sediment. Various effective devices are employed for this washing operation.

110. Control of Filter Operations. The most accurate way in which to control the operation of filter plants is to subject the water to a bacterial examination. This should be made at frequent intervals so as to note any possible changes in quality. The experience with European filter systems has shown that an impairment in quality has not infrequently been detected in time to prevent outbreaks of disease. In the larger filter plants, a bacteriological laboratory should be installed, and daily tests of the effluent made. The filter beds should be arranged so that the effluent from each can be tested separately, and provision made so that the filtered water can be rejected from any one filter if not up to standard. The careful control of the operations is a matter of great importance. In testing filters as to their efficiency, samples should be collected at periods when the effluent is likely to be the least favorable, as during frost periods, heavy rains, and periods of greatest consumption.

111. Results of Filtration. The most apparent result of filtration is in the removal of all the suspended matter in the water; but more important than this, a sand filter will remove very nearly all the bacteria originally present in the water. This is specially exemplified in the reduction of the deathrate from typhoid fever following upon the installation of a filter plant. But to secure continually good results it is essential that such works as filter plants be under efficient control.

RAPID FILTERS.

112. The *Rapid,* or as it is often called, the *Mechanical Filter,* is a form of filter designed to accomplish results in the way of purification comparable with those obtained by the slow sand filter already discussed, but with a much smaller sand area. It is like the sand filter in that the filtering material consists of a bed of 2 to 4 feet of sand or crushed quartz, but in other respects the construction and operation are widely different. The chief points are the very rapid rate of filtration (100 to 125 million gallons per acre per day), the use of a coagulant to aid in filtration, and the manner of washing the sand bed. Methods of operation and mechanical details are, to a large extent, covered by patents, and the filters are manufactured and sold by various filter companies.

Briefly, the filter consists of a wooden, steel or concrete tank in which the filtering material is placed and supported on a system of screens of various designs. In some forms the tanks are open and operated by gravity, and in others are closed and operated by pressure from the pipe system. When the filters become clogged through the accumulation of sediment on the surface, they are washed by forcing water in a reverse direction through the sand. During this process, the sand is, in most cases, agitated by means of mechanical agitators reaching deeply into the sand layer or by means of compressed air.

Two well-known types of filters are illustrated in Figs. 41 and 42. In the first, the water enters the settling chamber at the bottom, passes up through the central tube to the top of the filter and thence downwards through the sand to the collecting pipes located between filter and settling tank. Where a coagulant is used it is introduced before admission to the filter bed. The figure shows the agitators which are operated when the filter is washed. When not in use they are raised out of the sand. The wash water passes off on all sides over the top of the inner tank. In the other type the settling tank is not connected with the filter. No agitators are used; but to loosen the sand air is passed through the bed from below. In both forms shown the filtration is by gravity. The Warren filter is another type commonly employed. It uses agitators like the Jewell, but the settling tank is disconnected from the filter.

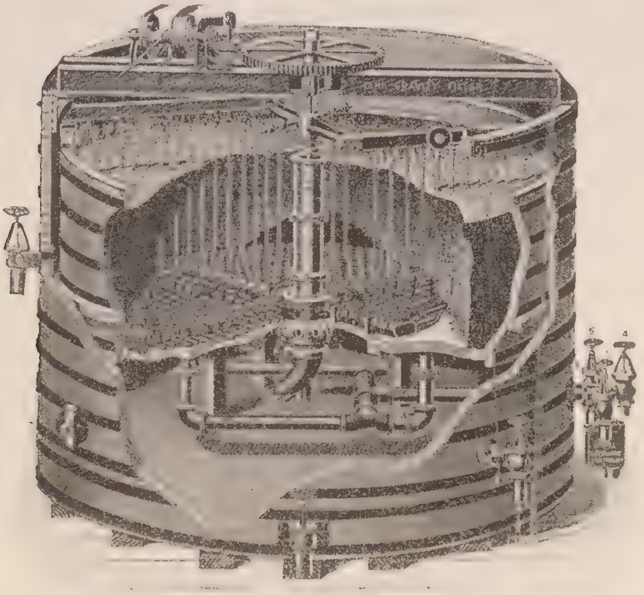


Fig. 41. Jewell Gravity Filter.

113. Principles of Operation. The action of mechanical filters is somewhat unlike that of sand filters, although the results are not greatly different. The effect of a coagulant in gathering the sediment into relatively large masses has been explained in section 101. It aids filtration in this way and also forms a substitute for the organic coating on the sand grains and on the surface of the ordinary sand filter. It is the use of the coagulant which enables such high velocities to be employed. To avoid too frequent washing, it is common to employ heads as high as 10 to 12 feet, but with such high heads and velocities the sand becomes clogged to a considerable depth. The methods of washing, however, enable this sediment to be readily removed. The interval between washings, *i.e.*, the "run," is 24 hours or less, and the operation of washing requires about 15 to 20 minutes. The amount of wash water used is commonly from 2 to 5 per cent of the applied water, which fact must be considered in determining the gross capacity of the plant. Crushed quartz is often used for the filtering material, but ordinary sand would probably do as well if of very uniform grain so as not to be carried away in the washing process. The coagulant employed is usually sulphate of alumina, but common alum is sometimes used. The relative merits of these and some other coagulants have been discussed in section 101. Ferrous sulphate is also employed as a coagulant with good results. Rapid filter plants were formerly installed by the companies who manufactured the mechanical devices, but some very complete plants have lately been designed and constructed under the supervision of consulting engineers. In this case some of the patented forms of strainers have been adopted and the other details been designed by the engineer in charge.

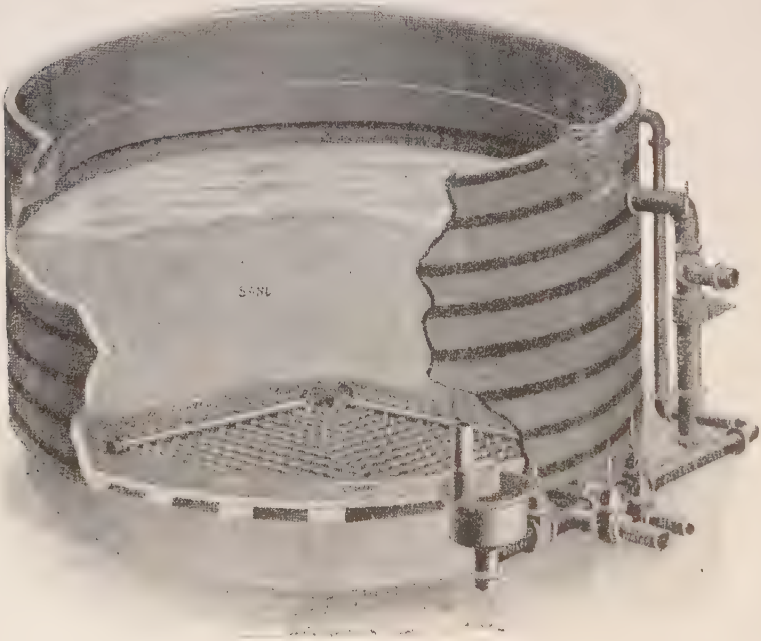


Fig. 42. Continental Gravity Filter.

In most of the early plants rapid filters have been adopted with particular reference to the removal of turbidity or color, but in some of the more recent plants the elimination of bacteria has been the chief object.

Extensive experiments with rapid filters indicate that when they are properly operated, turbidity can be practically all removed, a large percentage of color, and a considerable portion of dissolved organic matter. With the quantities of coagulant ordinarily used, they are probably slightly less efficient than sand filters in the removal of bacteria. In some places satisfactory clarification is obtained without a coagulant, but for good bacterial results a coagulant is necessary.

To obtain uniformly good results with economy requires very careful operation. The coagulant must be closely regulated to correspond with the quality of the water; in the case of waters low in alkalinity this is particularly necessary. The efficiency depends so entirely upon the control of these matters that the proper operation of a mechanical plant involves greater care on the part of the attendants than that of a sand filter. It is fully as important in this case also that the whole plant should be under the control of bacteriological tests, regularly and frequently made. Many points of operation, such as period between washings, wasting of water, thickness of sand layer, and best kind of sand, can be learned only after experience in the light of such analyses.

Considering the economic advantages of mechanical filters, it may be said that they are especially adapted to those cases where the cost of land is high, where the water is so turbid as to require large settling reservoirs with a sand filter plant, and in small plants where the unit for sand filters would be very small. They are also well adapted for the rapid removal of iron from ground waters or of the precipitate in softening plants. On the other hand, if the total annual expense of sand filters and mechanical filters is equal, the evidence points to the desirability of adopting sand filters, especially for sewage-laden waters, but the difference in efficiency is not great enough to warrant any considerable additional expense. With very turbid waters both systems are likely to prove unsatisfactory unless supplemented by adequate settling basins.

OTHER METHODS OF PURIFICATION.

114. Aeration. Aeration of a water, or the bringing of the water into close contact with the air, is useful in certain cases. It will have little or no effect upon organic matter present, but it does have a very important action in the case of waters coming from stagnant ponds and reservoirs in which putrefactive changes have taken place. Such waters will have offensive odors and tastes, due to the dissolved gases contained, and it is in the removal of these gases that the process of aeration can be successfully applied. It has been so used in a large number of cases.

Aeration may be accomplished by forcing air into the mains, or by passing the water over cascades or weirs, or by spraying it from a fountain into a reservoir as is done in many places, or by still other methods. The benefit of aeration explains why a well water raised by buckets is more commonly free from bad tastes and odors than where a pump is used, although such odors and tastes are not in themselves dangerous to the health.

115. Softening Water. Water is rendered hard by the presence of lime and magnesia, chiefly in the form of carbonates and sulphates, but occasionally as chlorids and nitrates. The carbonates cause so-called temporary hardness (removable by boiling), while the sulphates and other compounds cause permanent hardness. In using a hard water for washing purposes approximately 2 ounces of soap are neutralized or wasted for each 100 gallons of water for each grain per gallon of calcium carbonate or its equivalent. In boiler use the carbonates of lime and magnesia are precipitated, forming a deposit which can usually be removed by blowing out, unless accompanied by scale-forming substances. Sulphate of lime precipitates at high temperatures and forms a very hard, objectionable scale, particularly if the water contains other suspended matter. The softening of water is therefore of great economical importance.

The softening of water is accomplished by simple processes of chemical precipitation. To remove the carbonates, lime is used as the precipitant. The carbonates are held in solution chiefly by virtue of the carbonic acid dissolved in the water, and on adding lime the acid unites with it, forming carbonate of lime. In the case of hardness due to the carbonate of lime the reaction is

$$CaCO_3 + CO_2 + Ca(OH)_2 = 2CaCO_3 + H_2O.$$

The resulting carbonate is now but slightly soluble and so precipitates out.

The $CaCO_3$ (lime carbonate) and the CO_2 (carbonic acid) are present in the water; the $Ca(OH)_2$, ordinary lime, is the chemical added.

To remove the sulphate, sodium carbonate (Na_2CO_3) is used. The reaction is

$$CaSO_4 + Na_2CO_3 = CaCO_3 + Na_2SO_4.$$

The carbonate of lime precipitates out as before while the sodium sulphate (Na_2SO_4) is not especially objectionable. Various methods of carrying out the details of the process, relating principally to the application of the chemical and the removal of the precipitate, have been devised. These are known under various names, but the general principle is the same in all. The lime is usually added in the form of lime water, a solution of slaked lime in water.

In general the water to be treated is run into large tanks, the chemical added and then the precipitate allowed to settle as far as practicable. The water is then drawn off and the remaining precipitate removed by rapid filtration. In purifying water for boiler use the precipitate can be removed to a sufficient extent by the use of settling tanks alone. The chemicals used are lime and usually soda ash, or crude sodium carbonate. The cost for such small plants is reported to be from 4 to 15 cents per 1,000 gallons.

Many scale preventives have been proposed for use in boilers, but probably the best in general use is sodium carbonate. This breaks up the sulphates as previously shown, and thus prevents the formation of a hard deposit; but the precipitation of the carbonates is increased by the process. The sodium sulphate remains in solution, but should not be allowed to concentrate too greatly or it will cause foaming.

116. Domestic Filters. Frequently it is advisable to purify water supplies for household use. For this purpose a large number of different filters have been devised, but many of these are so inefficient as to be worse than useless; for it not infrequently happens that the possession of a filter lulls the consumer into a state of false security. The best of these filters suitable for household use are those that are made of unglazed porcelain (Pasteur filter) or fine infusorial earth (Berkefeld filter). Both of these filters deliver a wholly germ-free filtrate when they are first put in service, but unless close attention is given them they sooner or later lose this property. Generally speaking, these filters should be cleaned and sterilized in boiling water or in steam under pressure once a week in order to kill out the germ life that has found lodgment in the pores. In this way not only is the sterility of the filtrate maintained, but the yield of filtered water is increased.

Filters of this class are not often used for municipal purification, but are admirably adapted for schools, garrisons, prisons, or hotels as well as for private use.

Other types of household filters, such as those constructed of porous stone, charcoal, or asbestos, have been on the market for many years. Judged from the popular standpoint of purity, which is generally the production of a clear water, many of the filters would be regarded as quite satisfactory, but as a means of removing germ life they possess for the most part but little merit.

Another method on which even greater reliance can be placed is the use of heat. There are no pathogenic bacteria that are liable to be distributed by the way of the water supply that are able to withstand the influence of boiling water for a period exceeding 10 to 15 minutes. Cholera and typhoid succumb in 5 minutes or less. In case of sudden outbreaks of disease or temporary disturbance of installed water supplies, this method can always be relied on with perfect safety. Boiling does not, however, render potable a water containing large amounts of organic matter, although it may destroy the disease germs that may be therein. By distillation a water can be obtained free from dissolved matter as well as bacteria. This process is extensively used on shipboard to obtain potable water from sea water, and in a few places on the seacoast for similar purposes.

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